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NATIONAL DAM INSPECTION PROGRAM. LAKE ROLAND DAM. (NDI I.D. NUM--ETC(U)
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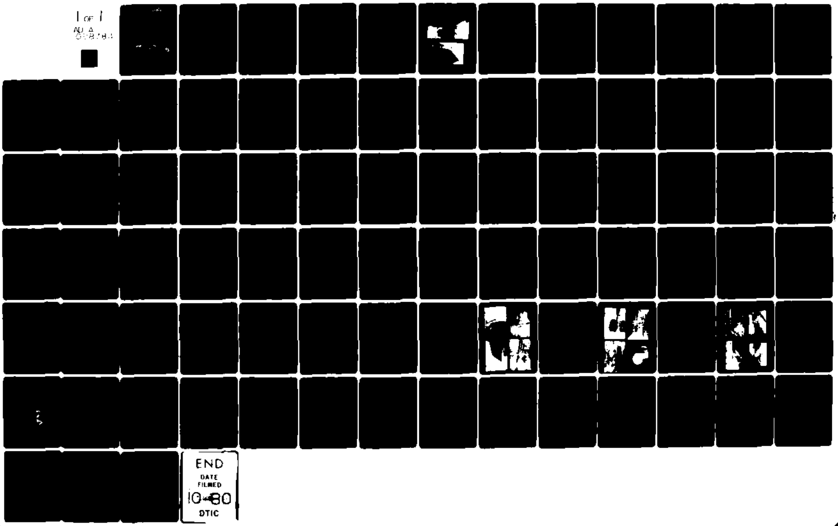

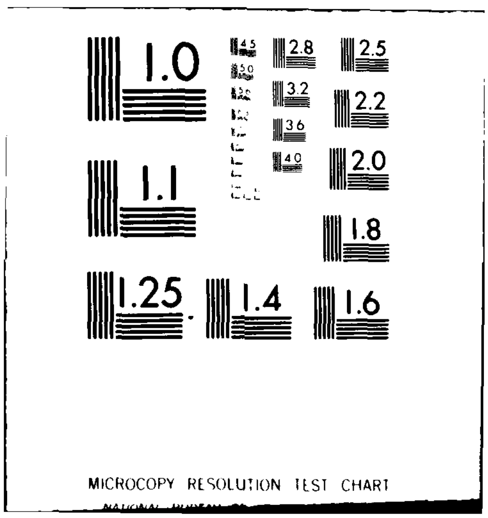
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CHESAPEAKE BAY BASIN
JONES FALLS
BALTIMORE COUNTY

MARYLAND

LAKE ROLAND DAM

NDI I.D. NO. MD 104

PHASE I INSPECTION REPORT

NATIONAL DAM INSPECTION PROGRAM.

Lake Roland Dam. (NDI I.D. Number MD-104), Chesapeake Bay Basin, Jones Falls, Baltimore County, Maryland.

PREPARED FOR

DEPARTMENT OF THE ARMY
BALTIMORE DISTRICT, CORPS OF ENGINEERS
BALTIMORE, MARYLAND 21203

DACW31-79-C-0038

BY

ACKENHEIL & ASSOCIATES, BALTIMORE, MD, INC.
7902 BELAIR ROAD
BALTIMORE, MARYLAND 21236

JULY 1979

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PREFACE

This report is prepared under guidance contained in the Recommended Guidelines for Safety Inspection of Dams, for Phase 1 investigations. Copies of these guidelines may be obtained from the Department of the Army, Office of Chief of Engineers, Washington, D.C. 20314.

The purpose of a Phase 1 investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general condition of the dam is based upon visual observations and review of available data. Detailed investigation and analyses involving topographic mapping, subsurface investigations, material testing, and detailed computational evaluations are beyond the scope of a Phase 1 investigation; however, the inspection is intended to identify any need for such studies which should be performed by the owner.

In reviewing this report, it should be realized that the reported condition of the dam is based on observations of field conditions at the time of inspection along with data avialable to the inpsection team. In cases where the reservoir was lowered or drained prior to inspection, such action, while improving the stability of the dam, removes the normal load on the structure and may obscure certain conditions which might otherwise be detectable if inspected under the normal operating environment of the structure.

It is important to note that the condition of the dam depends on numerous and constantly changing internal and external factors which are evolutionary in nature. It would be incorrect to assume that the present condition of the dam will continue to represent the condition of the dam at some point in the future. Only through frequent inspections can unsafe conditions be detected and only through continued care and maintenance can these conditions be prevented or corrected.

Phase 1 inspections are not intended to provide detailed hydrologic and hydraulic analyses. In accordance with the established Guidelines, the spillway design flood is based on the estimated "Probable Maximum Flood" (PMF) for the region (greatest reasonably possible storm runoff), or fractions thereof. The spillway design flood provides a measure of relative spillway capacity and serves as an aid in determining the need for more detailed hydrologic and hydraulic studies, considering the size of the dam, its general condition, and the downstream damage potential.

PHASE 1 REPORT
NATIONAL DAM INSPECTION PROGRAM

NAME OF DAM: Lake Roland Dam
STATE LOCATED: Maryland
COUNTY LOCATED: Baltimore
STREAM: Jones Falls, a tributary of the Patapsco River
DATES OF INSPECTION: March 15, 1979, and July 14, 1979
COORDINATES: Lat 39° 22.7', Long. 76° 38.6'

ASSESSMENT OF GENERAL CONDITIONS: Based on the evaluation of performance history, stability calculation results, and visual observations of conditions as they existed on the dates of the field reconnaissances, the general condition of Lake Roland Dam is considered to be fair. However, due to a seriously inadequate overflow section (spillway) the dam is categorized as "unsafe, non-emergency" in accordance with recommended criteria.

Lake Roland Dam is classified as an "intermediate" size, "high" hazard dam with a recommended spillway design flood of 100 percent PMF. Flood discharge capacity was found to be seriously inadequate based on the following data:

1) Non-overtopping flood discharge capacity is 10 percent PMF.

2) Failure of dam resulting from 35 percent PMF overtopping significantly increases the downstream loss of life and damage potential compared to that which would exist just before dam failure.

The reservoir drain slide gates are inoperable and judged inadequate in their present condition. The ability to drain the reservoir and perform remedial work on submerged portions of the dam requires that the reservoir drain be operational.

The following recommendations should be implemented as soon as possible:

1) Implement additional hydrologic and hydraulic studies to more accurately ascertain overflow section (spillway) adequacy and the extent of improvements required to provide sufficient discharge capacity or erosion/breaching protection for the dam. Dam improvements found necessary by the recommended study should be implemented immediately.

2) Repair and maintain reservoir drain slide gates and lifting mechanisms.

3) Develop a formal flood surveillance and warning plan.

4) Develop a more thorough inspection and maintenance program at the dam facility.

5) Remove tree located on the right (north) upstream abutment slope.

6) Repair abutment slope erosion and backfill animal burrows.

7) Replace and secure dislodged capping stones on spillway abutment walls.

8) Remove trees growing between stone block joints of the water supply outlet structure.

STATE OF MARYLAND
PROFESSIONAL ENGINEER

4 Aug 79
James D. Hainley, P.E. Date
MD Registration No. 528
Vice President

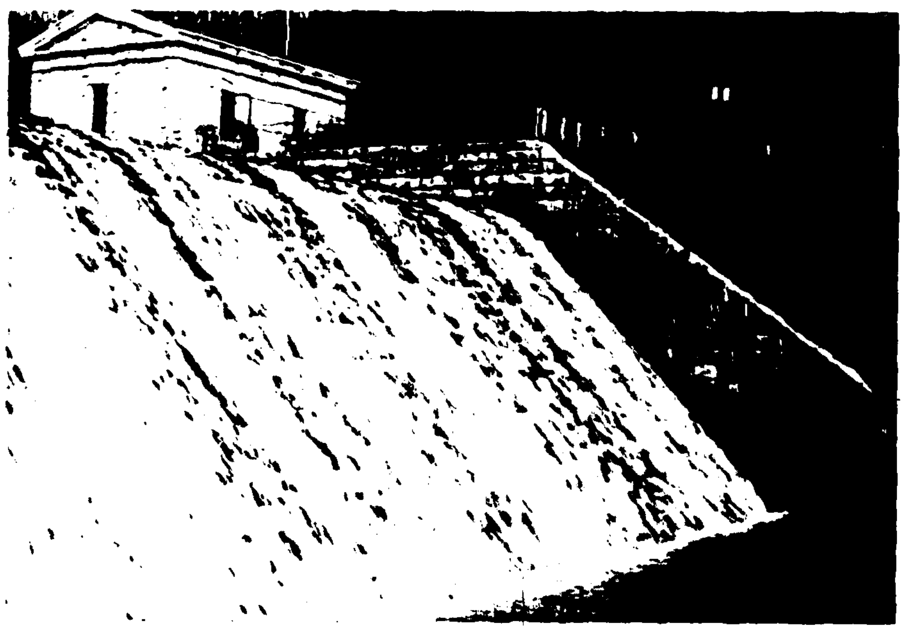
4 Aug 79
Timothy E. Debes Date
Project Engineer

4 Aug 79
Paul A. D'Amato Date
Project Engineer

APPROVED BY: 14 Sep 79
JAMES W. PECK Date
Colonel, Corps of Engineers
District Engineer

LAKE ROLAND DAM
BALTIMORE COUNTY, MARYLAND

OVERVIEW OF DAM



OVERFLOW SECTION AND INFLUENT GATE HOUSE

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PHASE 1 REPORT
NATIONAL DAM INSPECTION PROGRAM
LAKE ROLAND DAM
NATIONAL I.D. NO. MD 104

1.1 General

a. Authority. The study was performed pursuant to the authority granted by The National Dam Inspection Act, Public Law 92-367, to the Secretary of the Army, through the Corps of Engineers, to conduct inspections of dams throughout the United States.

b. Purpose. The purpose of this study is to evaluate if the dam constitutes a hazard to human life or property.

1.2 Description of Project

a. Dam and Appurtenances. The dam structure consists of an overflow section located between two non-overflow sections. (Refer to Drawing No. 2.)

1) Non-Overflow Sections. The non-overflow sections consist of two stone block walls with soil and rock backfill cover. The non-overflow sections extend from each side of the overflow section (spillway) located at mid-dam. The left (south) and right (north) non-overflow sections measure approximately 64 ft. and 126 ft., respectively. Maximum downstream toe to crest height is about 31 ft. The downstream non-overflow section slopes have 2H:1V inclinations.

2) Overflow Section and Appurtenances. Flood discharge facilities consist of an overflow section (spillway) located at mid-dam, and a round arch conduit which serves as a reservoir drain.

The 120 ft. wide overflow section has a 1H:2V upstream slope, 2.5H:1.5V downstream slope, and ogee shape crest. The crest is set at El. 225, six (6) ft. below the top of the non-overflow sections. Normal base flow and flood flows are discharged through the overflow section.

The reservoir drain consists of a round arch conduit measuring 6 ft. at the base, with 4 ft. high walls, and a round arch top section of 3 ft. radius. The reservoir drain conduit extends from the Influent Gate House to the left (south) overflow section sidewall, located 160 ft. downstream of the dam.

b. Location. Lake Roland Dam is located in Baltimore County, Maryland, approximately 0.45 mi. north of the city limits of Baltimore. The dam is situated on Jones Falls, a south flowing tributary of the Patapsco River.

c. Size Classification. Based on a maximum dam height of 31 ft. and a top of dam storage capacity of 1,867 ac.-ft. (excluding sediment storage), the dam facility is classified as an "intermediate" size structure.

d. Hazard Classification. Lake Roland Dam is located 0.45 mi. upstream from the city limits of Baltimore, Maryland. Substantial property damage and loss of life is expected to occur in the Jones Falls floodplain in the event of dam failure. The Jones Falls floodplain includes sections of the following communities: Bare Hills, Mount Washington, Village of Cross Keys, Woodberry, Hampden, and Baltimore City. The dam is therefore accordingly classified as a "high" hazard structure.

e. Ownership. Lake Roland Dam is owned by the City of Baltimore, Baltimore, Maryland. The Department of Public Works (Water Division) is responsible for the operation and maintenance of the slide gate lifting mechanisms located in the Influent Gate House. The Bureau of Parks and Recreation is responsible for the maintenance of Lake Roland Dam and reservoir.

f. Purpose of Dam. Lake Roland Dam and reservoir were originally intended to supply water for the City of Baltimore. However, its use for this purpose was abandoned in 1915. Since this time, Lake Roland has been primarily used for recreational purposes. Lake Roland Dam has a flood runoff storage capacity of about 867 ac.-ft.

g. Design and Construction History. Construction of Lake Roland Dam was started in 1860 according to dated construction drawings. The date, July 21, 1861, is discernible on an inscribed capping stone located at the right non-overflow dam section. This date is presumed to be the construction completion date. The Lake Roland-Hampden water supply conduit was sealed with a reinforced concrete plug on April 18, 1958.

h. Normal Operating Procedure. Lake Roland Dam operates as an uncontrolled structure and hence, does not require a dam tender. Under normal operating conditions, pool level is maintained at El. 225, the crest level of the uncontrolled overflow dam section.

1.3 Pertinent Data

a. Drainage Area — 36.8 sq. mi.

b. Discharge at Dam Facility

Maximum known flood at dam facility	Unknown
Ungated overflow section capacity at top of dam elevation	5,400 cfs

c. Elevation (feet above MSL)

Constructed top of dam	El. 231
Normal pool	El. 225
Overflow section crest	El. 225
Maximum tailwater	Unknown
Upstream invert of outlet pipe	El. 201
Downstream invert of outlet pipe	El. 200±
Streambed at toe of dam	El. 200±

d. Reservoir Length

Length of maximum pool	1.75 mi.
Length of normal pool	1.50 mi.

e. Total Storage

Constructed top of dam	1,867 ac.-ft.
Overflow section crest	1,000 ac.-ft.
Normal pool level	1,000 ac.-ft.
Sediment pool	Unknown

f. Reservoir Surface

Constructed top of dam	113 acres
Overflow section crest	100 acres
Normal pool	100 acres

g. Non-overflow Sections

Type	Stone masonry
Length	
Right section	126 ft.
Left section	64 ft.
Height	31 ft.
Side slopes	
Downstream	2H:1V
Upstream (submerged)	Unknown

h. Regulating Outlet

Type	Round arch conduit, stone block construction
Length of connecting outlet pipe	160 ft.
Gates	Two 4.5 dia. slide gates

i. Overflow Section

Type	Ogee
Width	120 ft.
Crest elevation	225 ft., MSL
Gate	None
Side slopes	
Downstream	1H:2V
Upstream	2.5H:1.5V

SECTION 2
DESIGN DATA

2.1 Design

a. Data Available. The following available data may be obtained from the Maryland Water Resources Administration or the City of Baltimore, Department of Public Works (Water Division).

1) Hydrology and Hydraulics. Unit and inflow flood hydrographs, and a summary of 50 year frequency and PMF peak inflows were obtained from Jones Falls Flood Control Study, Baltimore, Maryland. Study prepared June 1, 1971, for the City of Baltimore, Maryland by Knoerle, Bender, Stone & Associates, Inc.

2) Dam and Appurtenances. The available design data consists of as-built construction drawings obtained from City of Baltimore, Department of Public Works. These construction drawings include a centerline cross section and plan view of the non-overflow and overflow sections, and section views of the Influent Gate House and water supply conduit structure.

b. Design Features. Principal design features are illustrated on Drawing Nos. 2, 3, and 4.

1) Non-overflow Sections. Non-overflow wall sections are constructed of Cockeysville Marble stone blocks measuring approximately 3x2x1.5 ft. in dimension. Photographs of exposed stone wall sections indicate that block size and shape vary. A drawing showing a cross section view of the dam indicates that these wall sections are constructed on bedrock. The mortaréd stone block wall sections have an estimated base width of 20 ft. and an average height of 43 ft. These wall sections have a backfill cover consisting predominately of rock pieces mixed with soil. (Refer to Drawing No. 4.)

2) Overflow Section. The overflow section (spillway) is constructed of Cockeysville Marble stone blocks and extends to bedrock. The overflow section has an estimated maximum height from bedrock foundation to crest of 41 ft. A 16 ft. long stilling apron is located at the downstream toe.

2.2 Construction. Available design information is not sufficiently detailed to assess whether the dam and appurtenances were constructed in general accordance with intended design drawings and specifications.

2.3 Operation. The City of Baltimore, Department of Public Works (Water Division) is responsible for the operation of Lake Roland Dam. The only operational features at the dam are four (4) slide gates used to regulate flow entering the reservoir drain (round arch conduit) and water supply conduit. The slide gates are reportedly inoperable. No formal records of operation are maintained.

2.4 Evaluation

a. Availability. All available design information and drawings were obtained from the Dam Safety Division, Maryland Water Resources Administration and the City of Baltimore, Department of Public Works (Water Division).

b. Adequacy

1) Hydrology and Hydraulics. The available hydrological and hydraulic information is limited in scope. Computer analyses using HEC-1-DAM Safety Version were required to adequately conduct a Phase 1 study.

2) Dam and Appurtenances. The type and detail of available construction drawings and other data is limited in scope and number. This limited construction data required that assessments be heavily based on visual inspection, performance history, interpretation of photographs, and foundation, hydrologic, and hydraulic assumptions.

In view of the age of the dam (completed July 21, 1861), it is believed that the design approach and construction techniques are not likely to have been in conformance with currently accepted engineering practice. However, the performance history of the dam is reportedly good.

SECTION 3
VISUAL INSPECTION

3.1 Findings

a. General. The on-site reconnaissance of Lake Roland Dam consisted of:

1) Visual observation of non-overflow section slopes, reservoir, and downstream channel.

2) Visual observation of overflow section (spillway), overflow section sidewalls, water supply conduit structure, and reservoir drain outlet.

3) Visual observation of discernible hazardous conditions or safety deficiencies.

4) Evaluation of the downstream hazard potential.

A visual observation checklist and field sketch are given in Appendix A. Specific observations are illustrated in photographs of Appendix D.

In general, visual observations indicate the general condition of Lake Roland Dam is good. However, the dam is considered to be marginally maintained based on the inoperable condition of the reservoir drain slide gates and evidence of surficial deficiencies.

The following conditions were observed on the dates of the field reconnaissances:

b. Dam

1) Surficial. Downstream slopes of non-overflow sections are vegetated with grass. The right (north) downstream slope contains two (2) footpaths eroded into the grass cover approximately 20 and 100 ft. right (north) of the overflow section sidewall. A wide, shallow footpath is worn into the left (south) non-overflow section slope beside the overflow section sidewall.

The right (north) downstream non-overflow section slope is "pitted" in appearance. Two (2) animal burrows (possibly sink holes) are located at about mid-slope. A 2 ft. dia. tree is located on the upstream side of the right (north) non-overflow dam section.

2) Seepage/Wet Zones. On the dates of the field reconnaissances, seepage or surface wet zones were not discernible in the areas of the downstream slope or toe of the dam.

c. Appurtenant Structures

1) Overflow Section. Several capping stones are missing from both overflow section sidewalls. Reportedly, these capping stones were dislodged during Hurricane Agnes. Seepage was observed emanating from between stone blocks at several locations of both sidewalls. However, there was no apparent horizontal or vertical misalignment of these walls. Water turbulence, discernible mid-way between the overflow section crest and apron, suggests the possibility that one or more capping stones are misaligned.

2) Outlet Works. Outlet works consist of a plugged eliptical water supply conduit and a round arch stone block reservoir drain conduit. The lifting mechanisms, used to control conduit slide gates, are contained in the Influent Gate House. These slide gates and lifting mechanisms are reportedly inoperable.

Inspection of the water supply conduit structure indicates the reinforced concrete plug is in good condition. Water was present inside the conduit structure chamber and is presumed to be originating from the gated inlet. This impounded water partially drains through stone block joints of the outlet structure walls. Tree growth was observed extending from between stone block joints of the water supply outlet structure.

Water was also draining from the round arch conduit outlet (reservoir drain) located beside the left (south) overflow section sidewall. This drainage had an estimated flow rate of 10 gpm and is believed the result of leakage from the slide gates.

The right (north) exit stream channel bank is extensively eroded. This erosion extends from the right (north) overflow section sidewall to the single lane paved bridge, located 250 ft. downstream of the dam. The erosion extends from the stream channel to about 6 ft. up the stream bank slope.

d. Reservoir Area. Visual observations and a map review indicate reservoir slopes are predominately vegetated with woodland and some open field. Reservoir slopes and shoreline appear stable, exhibiting no evidence of landslides. However, urban development and slope erosion has contributed to a significant sediment-siltation problem in Lake Roland reservoir. An upstream investigation of Lake Roland reservoir found large quantities of silt and sediment deposited at the Roland Run, Towson Run, and Jones Falls stream inlets. These sediment deposits encompass the northern third of the reservoir. (Refer to Drawing No. 1.)

e. Downstream Channel. The downstream Jones Falls stream channel is about 60 ft. wide, cobble lined, and extends about 3,900 ft. before merging with a concrete lined channel. Falls Road and U. S. Interstate 83 overpass Jones Falls approximately 0.4 mi. and 1.7 mi. downstream of the dam, respectively. Approximately thirty (30) commercial and residential structures are located adjacent to and within a 20 ft. elevation difference of Jones Falls within a 1 mile channel reach.

3.2 Evaluation

a. Dam. The surficial deficiencies identified in Section 3.1 are not considered to represent significant hazard to the dam. However, embankment improvements should be made to backfill animal burrows and repair eroded footpaths. The tree located on the right (north) non-overflow section slope should be removed.

The general condition of the non-overflow sections is considered to be good.

b. Appurtenant Structures. The reservoir drain slide gates and lifting mechanisms are inoperable and judged inadequate in their present condition. Appropriate repairs should be made as soon as possible.

The capping stones dislodged from overflow section sidewalls during Hurricane Agnes should be replaced and secured. Most of these capping stones can be found downstream of the dam. Seepage emanating from between the stone block joints of these walls is not considered significant.

Trees growing between stone block joints of the water supply outlet structure walls should be removed to preserve structural integrity.

Erosion of the right (north) exit stream channel bank is not considered to affect dam stability.

SECTION 4
OPERATIONAL FEATURES

4.1 Procedure. The reservoir level is normally maintained at El. 225, the level of the ungated overflow section crest. Normal operating procedure does not require a dam tender. The gated water conduit and reservoir drain outlets are presently non-operational and remain closed.

4.2 Maintenance of Dam. The dam facility is maintained by the City of Baltimore. The Department of Public Works, Water Division is responsible for the maintenance of the gated water conduit and reservoir drain outlets. The Bureau of Parks and Recreation is responsible for maintenance of the dam and reservoir. Maintenance generally consists of cutting grass and removing trash and debris.

4.3 Inspection of Dam. There is no current record of formal inspections being conducted at the dam facility.

4.4 Maintenance of Operating Facilities. There is no record of how often the slide gate mechanisms of the reservoir drain are maintained and exercised. These slide gates were reported to be inoperable at the time of the field reconnaissance. According to the Department of Public Works, the water conduit is no longer in use and has been plugged.

4.5 Warning Systems in Effect. There is no warning system or formal emergency procedure to alert or evacuate, as necessary, downstream residents in the event or threat of a dam failure.

4.6 Evaluation. In general, maintenance procedures at Lake Roland Dam are considered marginal based on the observed surficial deficiencies and the inoperable condition of the slide gates. A more thorough maintenance program should be developed.

A formal inspection program should be instituted at the dam facility. In addition, a formal flood surveillance and warning plan is needed for the protection of downstream residents.

SECTION 5
HYDROLOGIC/HYDRAULICS

5.1 Evaluation of Features

a. Design Data. No hydraulic design data was available for the preparation of this report. The available unit and inflow hydrographs (50 year frequency, 0.6 hr., 1.0 hr., 1.5 hr., 3.0 hr., 6.0 hr., and 8.0 hr. storms) obtained from the Jones Falls Flood Control Study were of limited use. All calculation data used in this Phase 1 analysis was obtained by use of the U. S. Army Corps of Engineers Flood Hydrograph Package, HEC-1-DAM Safety Version.

b. Experience Data. Flood runoff, resulting from Hurricane Agnes (June 1972) reportedly overtopped Lake Roland Dam (non-overflow sections) by about 3 ft. The overtopping flood water severely eroded backfill from downstream non-overflow section slopes, exposing the stone block walls. Hurricane Connie, 1955, caused an estimated discharge through the overflow section of 5,500 cfs, which approximates maximum discharge capacity. The storm of September 10, 1968, resulted in overflow section discharges of about 3,600 cfs (reservoir pool level about 1.5 ft. below top of dam). These overflow section discharges were reported in the Jones Falls Flood Control Study.

As previously stated, Lake Roland Dam is classified as an "intermediate" size, "high" hazard dam. According to guideline criteria, the required spillway design flood for the dam facility is the PMF.

The PMF inflow hydrograph for the reservoir was modeled utilizing the HEC-1-DAM Safety Version computer program. This PMF inflow hydrograph was found to have a peak inflow rate of 61,500 cfs. Computer input and summary of output are included in Appendix C.

c. Visual Observations. On the dates of the field reconnaissances, no evidence of serious deficiencies or conditions were observed that would significantly reduce overflow section (spillway) discharge capacity in the event of a flood. The inoperable slide gates will not significantly decrease flood discharge capacity.

d. Overtopping Potential. Various percentages of PMF rainfall were routed through the reservoir to estimate the percent PMF inflow that the overflow section can pass without overtopping the dam. The computer analyses indicate the overflow section can pass approximately 10 percent PMF without overtopping the dam. Computer analysis results also indicate ½ PMF and PMF runoff overtop Lake Roland Dam by maximum depths of about 7 and 12 ft., with flow durations of about 9 and 10.5 hours, respectively.

e. Adequacy of Overflow Section

1) General. Adequacy of the overflow section was evaluated in accordance with procedures and guidelines established by the U. S. Army Corps of Engineers for Phase 1 hydrologic and hydraulic studies.

As previously reported, the overflow section does not have adequate capacity to pass the recommended spillway design flood of 100 percent PMF without overtopping the dam. The dam is overtopped by runoff resulting from rainfall in excess of 10 percent PMF. Guideline criteria requires an estimation be made of the likelihood of dam failure, and downstream damage and loss of life consequences for dams overtopped by less than ½ PMF conditions.

The HEC-1-DAM Safety Version computer program was used to evaluate breaching of the dam, and estimate the downstream hydrologic/hydraulic consequences resulting from assumed structural failure(s). This data is required to assess if the overflow section discharge capacity is seriously inadequate.

2) Analysis. A breach analysis was conducted to estimate if dam failure resulting from overtopping would significantly increase loss of life or damage downstream from the dam compared to what would exist just before dam failure. This analysis was performed in three steps.

In the first step, the percent PMF inflow that would initiate breaching was selected based on the performance history of Lake Roland Dam during Hurricane Agnes and stability calculation results presented in Section 6. Photograph Nos. 9, 10, 11, and 12 (Appendix D) show the erosion damage of non-overflow section slopes resulting from Hurricane Agnes. Overtopping flows reportedly reached a maximum stage level of 3 ft. above top of dam (El. 234). Computer analyses indicate a corresponding overtopping flow duration of about 6 hours and a maximum overtopping depth of 3 ft. for approximately a 20 percent PMF storm. (Refer to Appendix page C-4.) The photographs indicate the right (north) non-overflow section was subject to the worst erosive damage during Hurricane Agnes. Based on this information, the right (north) non-overflow section was considered the most likely to fail when the dam is overtopped.

Stability calculation results (presented in Section 6) indicate failure of the right (north) non-overflow section can be expected when overtopping flows reach about 5 ft. Failure was based on the assumption that downstream backfill cover of non-overflow sections will be eroded to about bedrock level (El. 188±). The 5 ft. overtopping required for failure approximately corresponds to a 35 percent PMF inflow, which was selected as the "failure" storm.

In the second step, the selected 35 percent PMF inflow was routed through the reservoir and downstream potential damage centers to estimate flood stage levels prior to incipient failure of the dam. This flood stage level is to serve as a reference for comparison, to estimate if breach flood levels caused a significant increase in the downstream hazard. (Plan 1, computer output.) (Refer to Drawing No. 5 and Appendix E for cross sections and location of damage center stations.)

Finally, breach flood stages in the potential damage areas were estimated by routing the 35 percent PMF inflow combined with the discharge that would be contributed by failure of Lake Roland Dam. The breach analysis was based on the following:

a) Depth of maximum overtopping prior to failure: 4.5 ft.

b) Duration of overtopping prior to failure: about 3 hrs.

c) Breach section width of 126 ft. and height of 24 ft. (Plan 4, computer input.)

d) Duration of failure: 0.10 hr.

3) Results. Drawing No. 5 illustrates cross sections of two (2) damage center stations evaluated in the analysis. These cross sections show the increase in water surface elevation which results from the assumed structural failure mode considered by the analysis.

Review of the flood stages before and after dam failure indicates that flood stages would be raised by about 8.7 ft. at Sta. 3 and 8.4 ft. at Sta. 4. This rise in flood stage due to dam failure is considered to significantly increase the loss of life and downstream damage potential. Therefore, the discharge capacity of the overflow section is considered to be seriously inadequate.

SECTION 6
STRUCTURAL STABILITY

6.1 Evaluation of Structural Stability

a. Visual Observations. The structural condition of Lake Roland Dam is assessed as good at the present time. No significant structural deficiencies were noted during the field reconnaissance. The tree, located on the right (north) non-overflow dam section, may cause structural damage if allowed to continue to grow.

b. Design and Construction Data. Available construction data includes as-built drawings showing a plan view and longitudinal cross section of the dam. Photographs were also available showing downstream sections of exposed non-overflow section walls where soil backfill had been eroded by flood flows from Hurricane Agnes (June 1972). The following information was obtained from these drawings and photographs.

1) Geometry of dam as shown in Drawing Nos. 2, 3, and 4.

2) The dam is founded on bedrock.

3) Stone blocks used for dam construction have typical dimension of 3x2x1.5 ft.

No other design or construction data was available for use in evaluating structural stability.

c. Performance Data. Photographs and conversation with City of Baltimore officials concerning performance of Lake Roland Dam during Hurricane Agnes (see Photograph Nos. 9, 10, 11, and 12) indicate the following:

1) Flood flows overtopped the dam (non-overflow sections) by approximately 3 ft.

2) Soil backfill downstream of non-overflow sections was eroded to about El. 210.

3) Stone block portions of the dam were not significantly damaged. However, some capping stones were dislodged by flood flows.

There is no report of the dam ever having failed since its construction in 1861.

d. Stability Analyses. Due to lack of design data, an analysis was conducted to evaluate the stability of the dam. Geometry of overflow and non-overflow sections was obtained from as-built drawings and/or interpreted from photographs. The drawings indicate that the dam rests on bedrock. In order to perform a stability analysis, it was necessary to make the following assumptions:

1) Non-overflow sections of the dam will fail before the overflow section. The overflow section is more massive in construction and has an estimated base width of 3 times the width of non-overflow sections. (See Drawing Nos. 3 and 4.)

2) Unit weight of stone blocks equals 168 pcf (typical value for marble).

3) Soil backfill downstream of non-overflow sections has a unit weight of 122.4 pcf and strength parameters of $\phi = 25^{o}$, c = 200 psf.

4) Coefficient of friction between stone blocks, and between stone blocks and bedrock equals 0.65. (Ref. Vector Mechanics for Engineers: Statics and Dynamics, Beer and Johnston, 1972.)

5) Reservoir depth at dam about 18 ft. (based on soundings taken in 1975).

6) Sediment behind dam has a specific gravity of 1.45, c = 0, $\phi = 0^{o}$.

7) Drag forces caused by flow of water over the dam are negligible.

8) Stone blocks are mortared in place (i.e. dam acts as a rigid body).

9) One hundred percent of uplift force acts at base of dam and has trapezoidal pressure distribution.

10) A k_o value equal to 0.8 was used to analyze at-rest lateral earth pressure of backfill downstream of non-overflow dam sections.

Analysis No. 1 - Agnes Storm. The validity of the above assumptions was tested by considering the stability of the dam during overtopping by Hurricane Agnes. During Agnes, as previously stated, the dam proved stable when subject to 3 ft. of overtopping and erosion of non-overflow section backfill to about El. 210. Considering the dam to act as a rigid body, the following factors of safety were respectively computed for sliding and overturning for the above condition and assumptions:

Sliding at Base	
Passive Condition*	F.S. = 1.43
At Rest Condition	F.S. = 0.90
Overturning	
Passive Condition*	F.S. = 1.18
At Rest Condition	F.S. = 0.97

*Passive lateral earth pressure of backfill downstream of non-overflow dam sections based on Rankine empirical formulas for comparison purposes.

These factors of safety indicated the dam was close to failure but should not have failed. (See Appendix G, Analysis No. 1.)

Analysis No. 2 - Present Condition. Analysis (see Appendix G) was conducted to evaluate the stability of the dam in its present structural condition. The analysis was conducted with reservoir level at top of dam, complete soil cover downstream of non-overflow sections, and with the same assumptions used to analyze Hurricane Agnes conditions. This analysis yielded the following factors of safety for sliding and for overturning:

Sliding at Base	
Passive Condition*	F.S. = 2.83
At Rest Condition	F.S. = 2.30
Overturning	
Passive Condition*	F.S. = 2.30
At Rest Condition	F.S. = 1.40

*Passive lateral earth pressure of backfill downstream of non-overflow dam sections based on Rankine empirical formulas for comparison purposes.

These factors of safety do not meet the recommended 3.0 minimum criteria for static conditions.

Analysis No. 3 - 5 ft. Overtopping 35% PMF. Further analysis was performed, assuming that 5 ft. of overtopping would cause complete removal of non-overflow section backfill (see Appendix G, Analysis No. 3). Hydrologic/hydraulic analysis of PMF storm conditions indicate that the non-overflow sections would be overtopped by as much as 12 ft. Complete erosion of non-overflow section backfill at some stage of overtopping was therefore considered likely. The following factors of safety were respectively computed for sliding and overturning:

Sliding at Base	F.S. = 0.46
Overturning	F.S. = 0.75

The non-overflow sections are thus considered unstable if soil rock backfill is completely eroded, and overtopping flows approximate 5 ft.

e. Operating Records. Operating records are not maintained at the dam facility.

f. Post-Construction Changes. A water supply conduit was constructed in 1885. This conduit was regulated by slide gates located in the gate house (see Photograph No. 6). The conduit was plugged in 1958 and is now inoperable.

According to Bureau of Parks and Recreation personnel, small rock pieces were mixed with soil, and placed on the upstream (to about 5 ft. below normal pool) and downstream non-overflow section slopes after Hurricane Agnes (June 1972).

g. <u>Seismic Stability</u>. Analysis No. 4 (earthquake - present condition) was conducted in order to evaluate the stability of the non-overflow sections under earthquake conditions (see Appendix G). This condition was analyzed with reservoir level at top of dam, complete soil cover on non-overflow sections (same as Analysis No. 2), and horizontal and vertical acceleration of 0.025 g (Seismic Zone 1). The following factors of safety were respectively computed for sliding and overturning:

Sliding at Base	
Passive Condition*	F.S. = 2.60
At Rest Condition	F.S. = 1.33
Overturning	
Passive Condition*	F.S. = 2.07
At Rest Condition	F.S. = 1.28

*Passive lateral earth pressure of backfill downstream of non-overflow dam sections based on Rankine empirical formulas for comparison purposes.

The computed at rest condition factors of safety do not meet the recommended 1.5 minimum criteria for earthquake conditions. Also, as indicated in the static analysis, the dam is considered unstable if overtopping completely erodes non-overflow section backfill.

h. <u>Location of Resultant</u>. Analysis No. 5 (non-seismic, present condition) was conducted to estimate the location of the resultant of all forces acting on non-overflow section walls. This condition was analyzed with reservoir level at top of dam and complete soil cover on non-overflow section walls (same as Analysis No. 2). Moment calculations indicate passive and at rest condition resultant forces act through the middle third of the base, and are therefore in agreement with recommended guidelines.

SECTION 7
ASSESSMENT, RECOMMENDATIONS/PROPOSED REMEDIAL MEASURES

7.1 Dam Assessment

a. Evaluation

1) Dam. Lake Roland Dam is considered to be in fair condition at the present time. This conclusion is based on performance history, stability calculation results, and visual observations of conditions as they existed on the dates of the field reconnaissances.

2) Reservoir Siltation. Visual observations and available data indicate large quantities of sediment are deposited in Lake Roland reservoir. This deposition of sediment is believed attributable to urban construction within the watershed, and subsequent transporting of disturbed surface soils by surface drainage. The deposition of excessive quantities of sediment has transformed the upstream reaches of Lake Roland reservoir into a shallow, swamp-like area.

3) Slide Gates. The reservoir drain slide gates are inoperable and judged inadequate in their present condition. The ability to drain the reservoir and perform remedial work on submerged portions of the dam requires that the reservoir drain be operational.

4) Structural Stability. In general, Lake Roland Dam was found to have inadequate stability under static and Seismic Zone 1 earthquake conditions based on recommended criteria. As indicated by static analysis, the dam is considered unstable if overtopping erodes non-overflow section backfill.

5) Overtopping Potential. Hydrologic/hydraulic analyses indicate that the dam can pass runoff (5,397 cfs) resulting from about 10 percent PMF (2.5 in./6 hr.) without being overtopped. This rainfall amount is less than the recommended spillway design flood of 100 percent PMF required by the size and hazard classification of the dam. Computer analyses indicate PMF inflow will cause a 12 ft. overtopping of the dam.

6) Adequacy of the Overflow Section. As presented in Section 5, overtopping of the dam by 35 percent PMF inflow is reasonably expected to cause failure based on stability calculations. HEC-1-DAM Safety Version computer analyses indicate downstream flood stage levels would be raised by about 8.5 ft. in the event of the assumed dam failure. This rise in flood stage is considered to significantly increase the loss of life and downstream damage potential. Therefore, the discharge capacity of the overflow section is considered to be seriously inadequate. The dam is categorized as "unsafe, non-emergency", based on guideline criteria.

b. Adequacy of Information. The construction drawings and other data available for this review were limited in scope and detail. Assessment of dam condition was based on this data, visual observations, performance history, interpretation of photographs, and construction, hydrological and hydraulic assumptions.

c. Necessity for Further Investigation. The owner should initiate additional studies to more accurately ascertain overflow section adequacy and the extent of improvements required to provide sufficient discharge capacity or erosion/breaching protection for the dam.

d. Urgency. The recommendations/remedial measures presented in this report should be implemented as soon as possible.

7.2 Recommendations/Remedial Measures. The following recommendations are presented based on the data obtained:

a. Dam

1) Implement additional studies to more accurately ascertain overflow section adequacy and the extent of improvements required to provide sufficient discharge capacity or erosion/breaching protection for the dam. Dam improvements found necessary by the recommended study should be implemented immediately.

2) Repair and maintain reservoir drain slide gates and lifting mechanisms.

3) Remove tree located on the right (north) upstream non-overflow section slope.

4) Repair erosion and backfill animal burrows on downstream non-overflow section slopes.

5) Replace and secure dislodged capping stones on sidewalls of overflow section.

6) Remove trees growing between stone block joints of the water supply outlet structure.

b. Operation and Maintenance Procedures

1) Develop a formal flood surveillance and warning plan. Plan to include, but not limited to, the following:

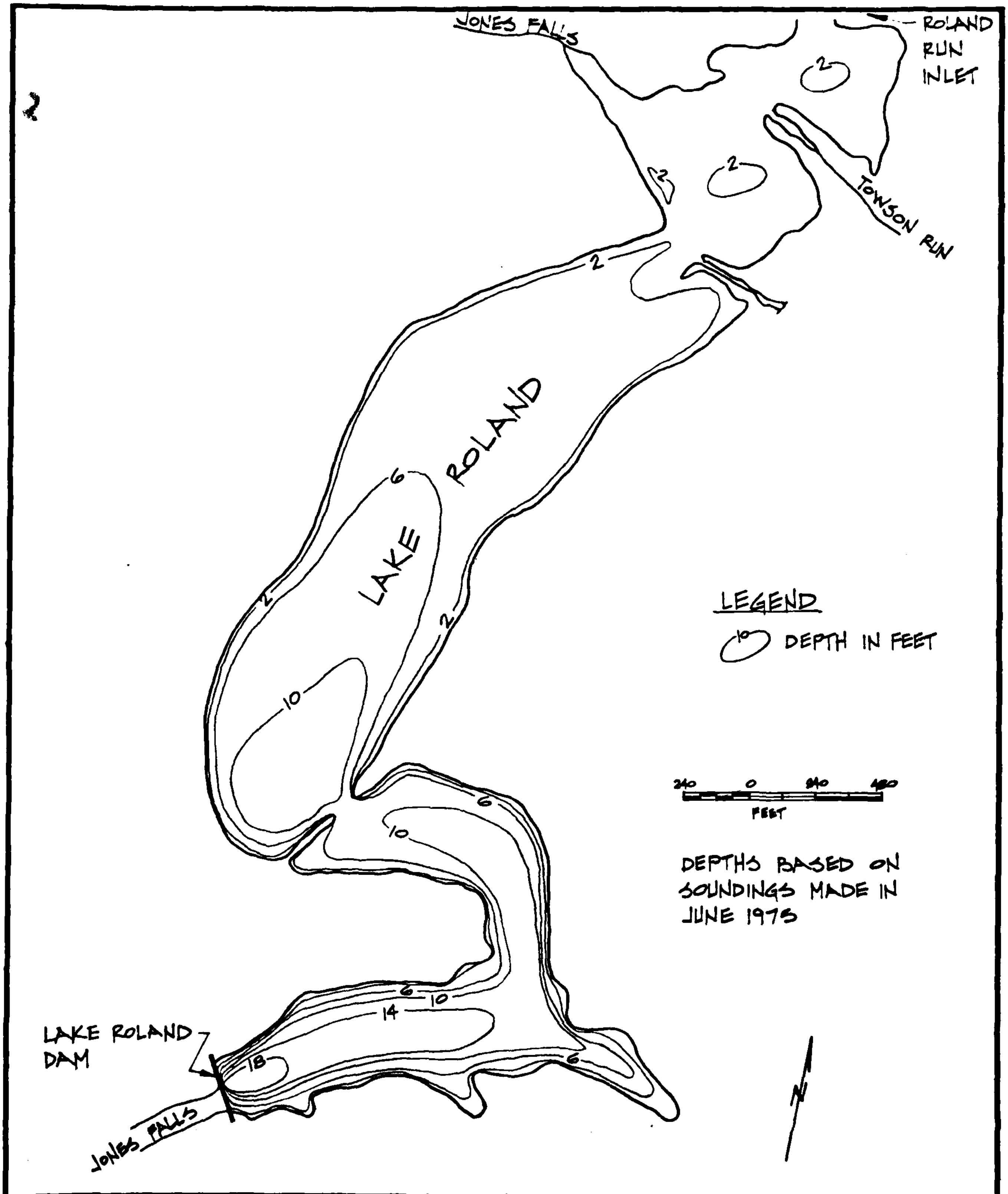
a) Surveillance. Around-the-clock surveillance of overflow section discharge and overtopping of dam during periods of unusually heavy rainfall.

b) Warning System. Formal warning procedures to alert downstream residents in the event of expected high flood flows.

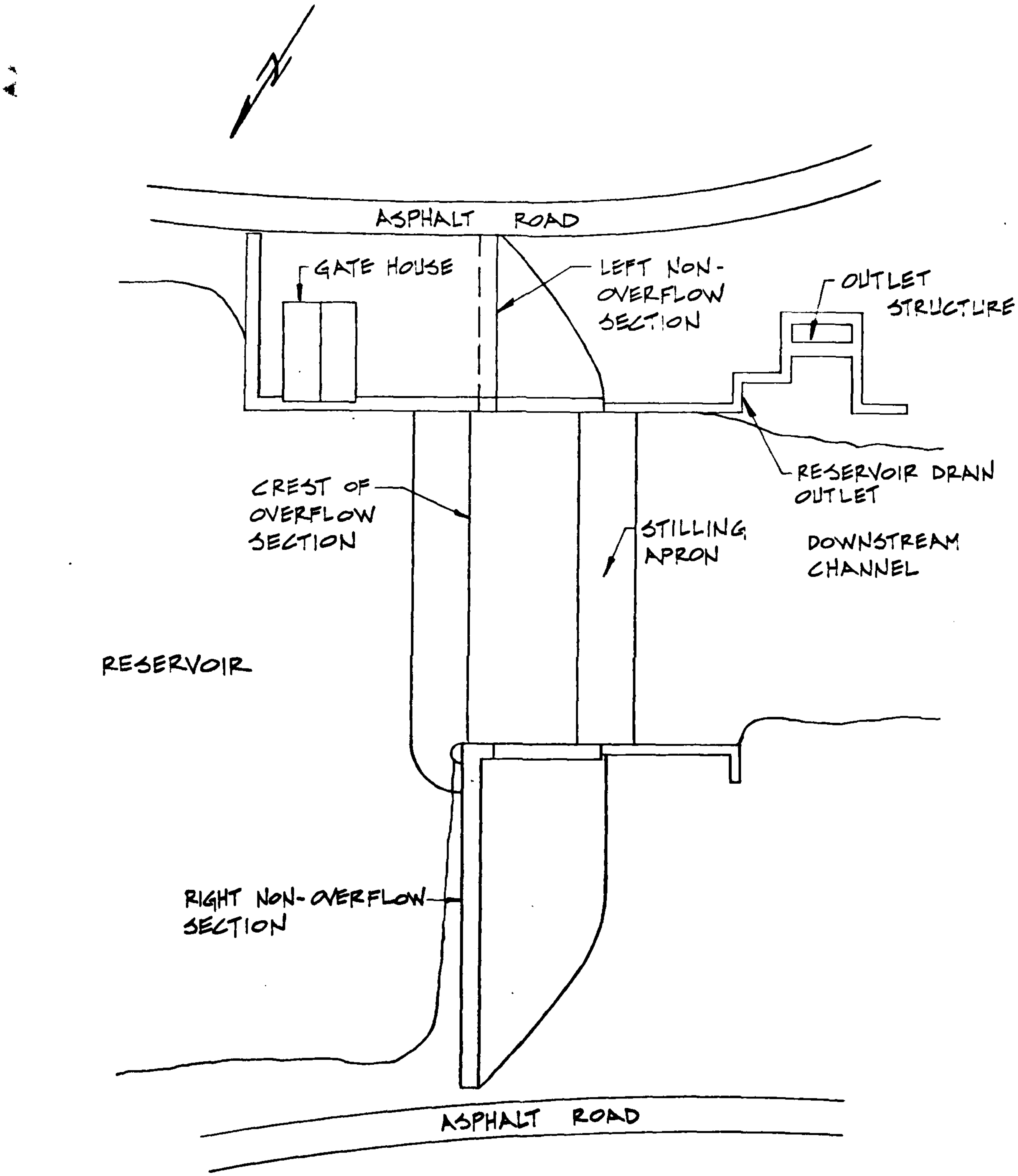
c) <u>Evacuation Plans</u>. Adequate emergency contingency plans to evacuate downstream residents in the event or threat of a dam failure.

2) Develop a more thorough inspection and maintenance program at the dam facility. Maintenance program should include frequent maintenance and exercising of the reservoir drain slide gates and prompt remedial treatment of deficiencies.

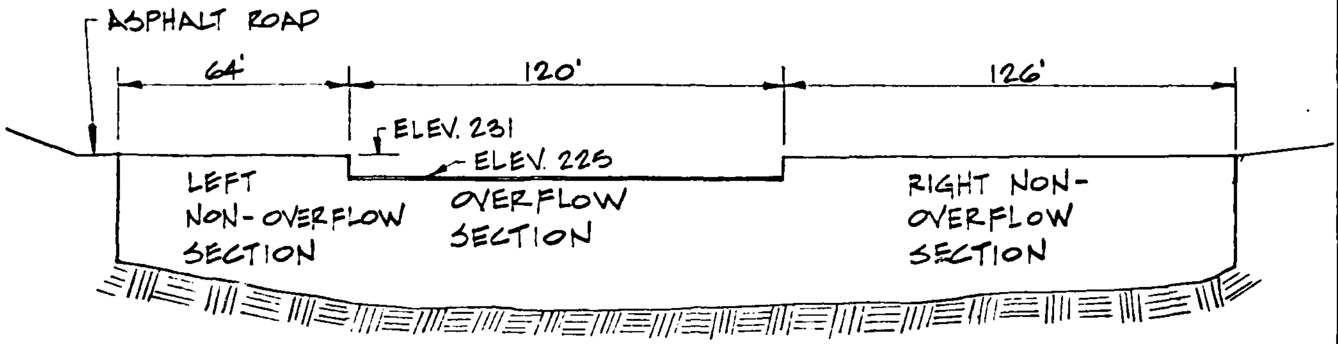
DRAWINGS



TE: JULY 13, 1979	NATIONAL DAM INSPECTION PROGRAM	LAKE ROLAND
SCALE: AS SHOWN		
DR: JLM — CK: T.E.D.	ACKENHEIL & ASSOCIATES CONSULTING ENGINEERS BALTIMORE, MD.	
DWG. NO. 1		

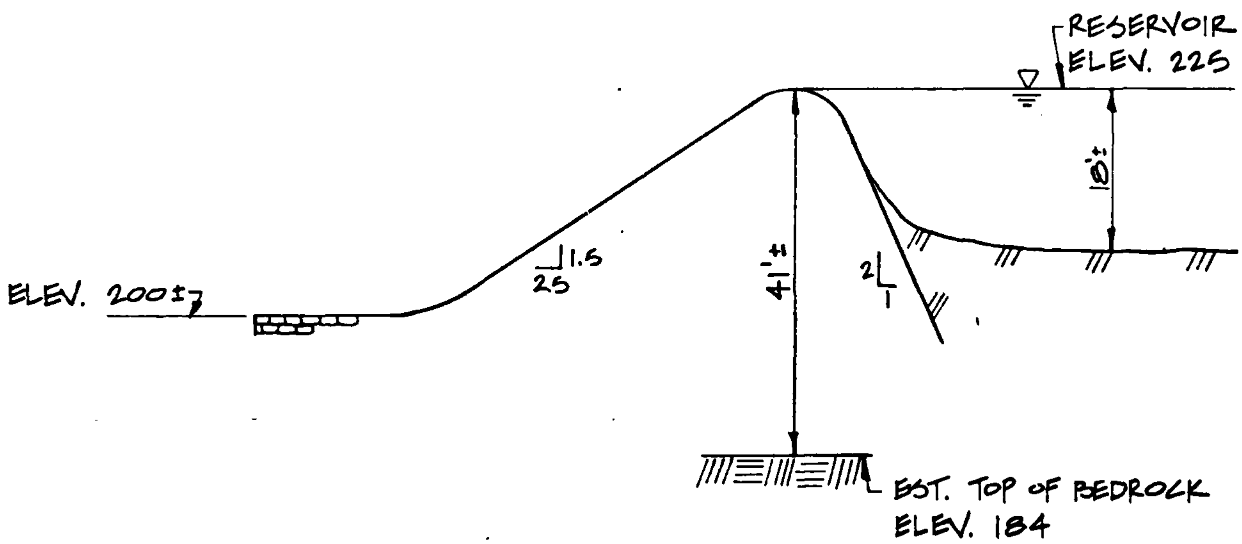


TE: JULY 13, 1979		NATIONAL DAM INSPECTION PROGRAM	PLAN VIEW OF LAKE ROLAND DAM
SCALE: NONE			
DR: DEH	CK: T.E.D.	ACKENHEIL & ASSOCIATES CONSULTING ENGINEERS BALTIMORE, MD.	
DWG. NO. 2			



SECTION THRU DAM LOOKING DOWNSTREAM

1" = 50'

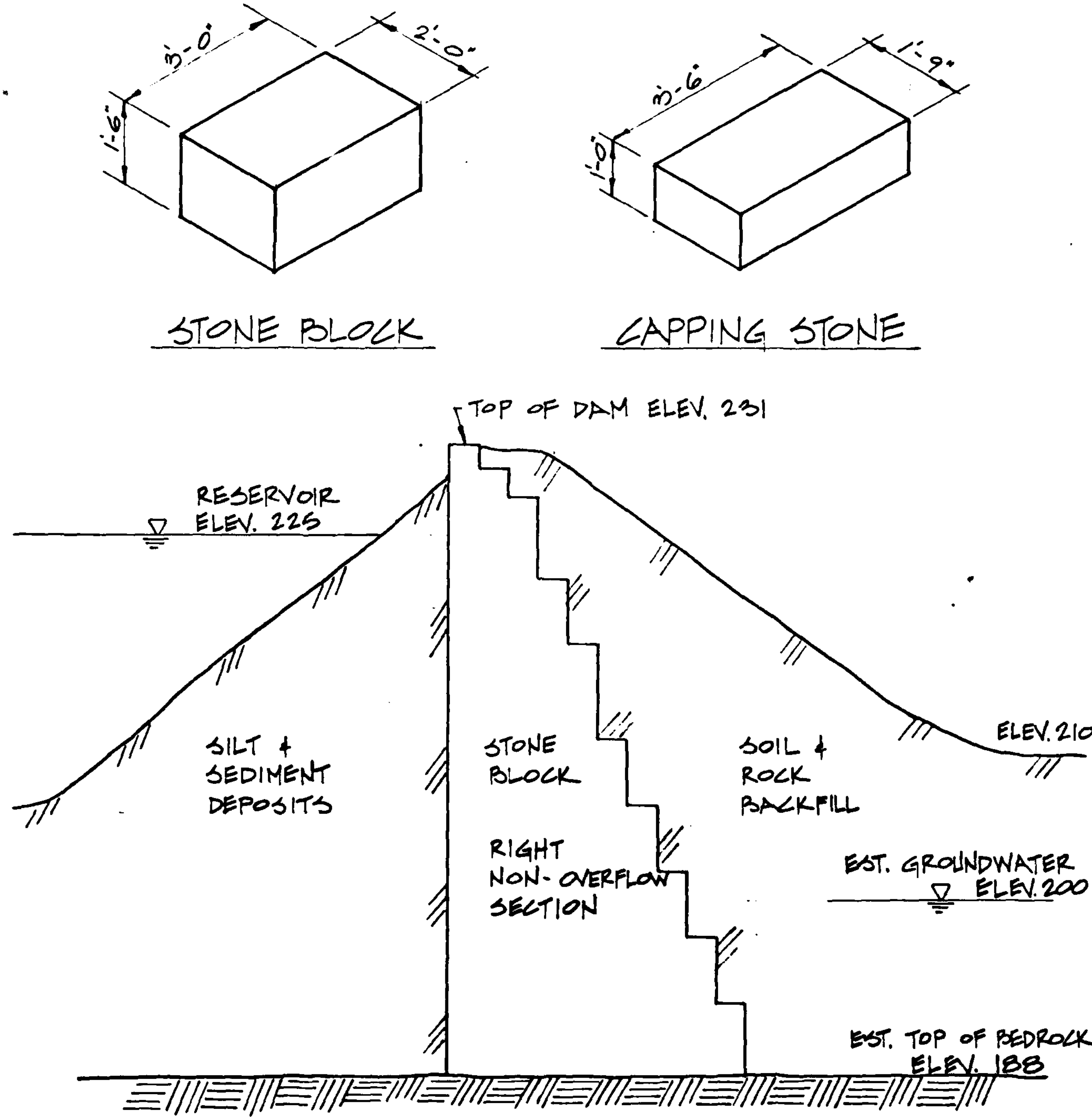


SECTION THRU OVERFLOW SECTION

1" = 20'

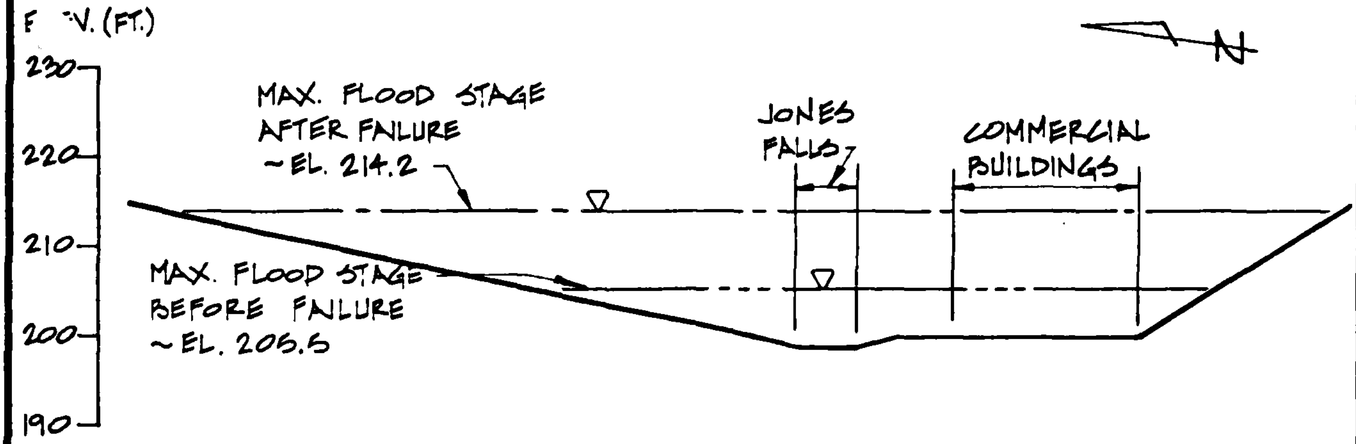
NOTE: CROSS SECTIONS INTERPRETED FROM FIELD MEASUREMENTS, PHOTOGRAPHS AND AS BUILT PLAN VIEW DWG.

DATE: JULY 13, 1979		NATIONAL DAM INSPECTION PROGRAM	DAM CROSS SECTIONS
SCALE: AS SHOWN			
DR: JLM	CK: T.E.D.	ACKENHEIL & ASSOCIATES CONSULTING ENGINEERS BALTIMORE, MD.	
DWG. NO. 3			

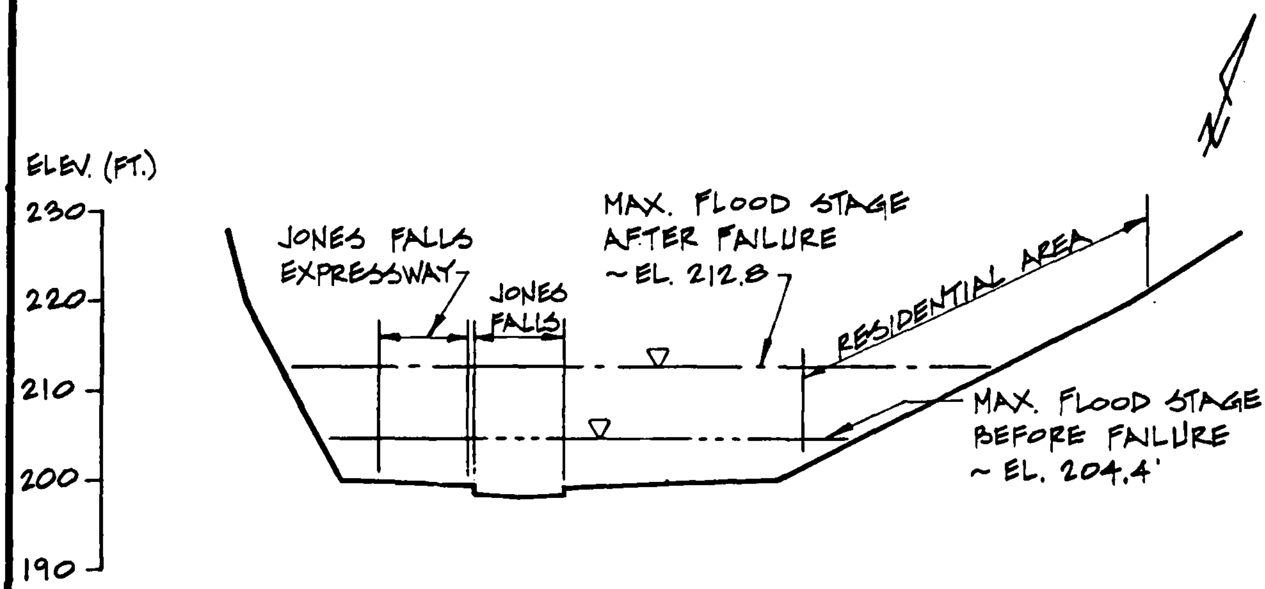


NOTE:
GEOMETRY OF NON-OVERFLOW SECTION INTERPRETED FROM PHOTOGRAPHS

DATE: JULY 13, 1979		NATIONAL DAM INSPECTION PROGRAM	CROSS SECTION OF RIGHT NON-OVERFLOW DAM SECTION
SCALE: 1" = 10'			
DR: JLM	CK: T.E.D	ACKENHEIL & ASSOCIATES CONSULTING ENGINEERS BALTIMORE, MD.	
DWG. NO. 4			



SECTION @ STATION 3 (2,000' FROM DAM)
VERT. 1" = 20' HORZ. 1" = 200'



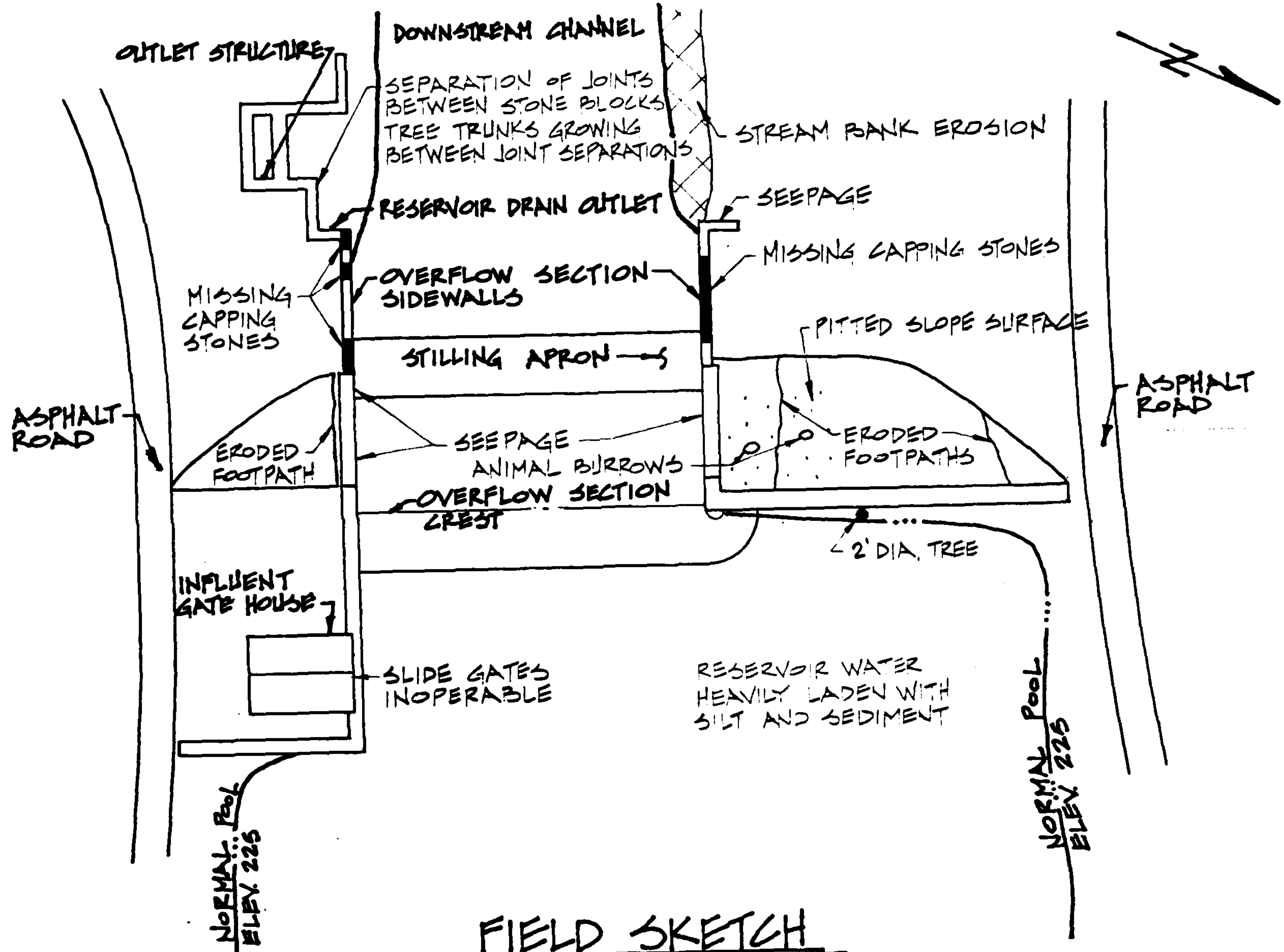
SECTION @ STATION 4 (4,000' FROM DAM)
VERT. 1" = 20' HORZ. 1" = 200'

NOTE:
SECTIONS WERE DEVELOPED FROM U.S.G.S. TOPOGRAPHY AND ARE ONLY APPROXIMATE.

DATE: JULY 13, 1979		NATIONAL DAM INSPECTION PROGRAM	CROSS SECTIONS OF DAMAGE CENTERS
SCALE: AS SHOWN			
DR: JLM	CK: T.E.D.	ACKENHEIL & ASSOCIATES CONSULTING ENGINEERS BALTIMORE, MD.	
DWG. NO. 5			

APPENDIX A

FIELD SKETCH AND VISUAL OBSERVATIONS CHECKLIST



FIELD SKETCH
LAKE ROLAND DAM

A1

VISUAL OBSERVATION CHECKLIST

Name Dam Lake Roland Dam County Baltimore State Maryland National ID # MD 104

Type of Dam Stone-Masonry Hazard Category High - Class 1

Date(s) Inspection 3/15/79 Weather Clear Temperature 45^{o} F

Inspection Review Date 7/14/79 (Ackenheil & Associates personnel only.)

Pool Elevation at Time of Inspection 225* Tailwater at Time of Inspection Normal M.S.L.

*Pool at overflow section crest level.

Inspection Personnel:

Ackenheil & Associates	Water Resources Administration	Baltimore City Officials
Timothy Debes	Jeffrey Smith	Bennett Chalk
James Hainley	Douglas Moore	Harold Fennel
John Huang	Thomas Moynahan	Ernest Grimm
Richard Gabell		Edward Schneider

Recorder Timothy Debes

CONCRETE/MASONRY DAMS

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SURFACE CRACKS CONCRETE SURFACES	No significant cracking of stone blocks observed.	
STRUCTURAL CRACKING	None noted, except as indicated in "Construction Joints" section.	
VERTICAL AND HORIZONTAL ALIGNMENT	No significant vertical or horizontal misalignment of overflow or non-overflow sections.	
MONOLITH JOINTS	N/A	
CONSTRUCTION JOINTS	Separation of stone block joints observed at downstream end wall of left overflow section sidewall.	
STAFF GAGE AND RECORDER	None.	

CONCRETE/MASONRY DAMS

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
ANY NOTICEABLE SEEPAGE	Small quantities of seepage emanating between stone blocks at overflow section sidewalls and water supply conduit structure walls.	
STRUCTURE TO ABUTMENT/EMBANKMENT JUNCTIONS	Abutment junctions stable.	
DRAINS	None observed.	
WATER PASSAGES	1. Round arch conduit (reservoir drain) exits at the downstream end wall of the left overflow section sidewall. 2. Water supply conduit: abandoned 1915, plugged 1958.	
FOUNDATION	Overflow and non-overflow dam sections and water supply conduit structure built on bedrock.	

NON-OVERFLOW SECTION SLOPES

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS*
SURFACE CRACKS	None observed.	
UNUSUAL MOVEMENT OR CRACKING AT OR BEYOND THE TOE	None observed.	
SLOUGHING OR EROSION OF NON-OVERFLOW SECTION SLOPES	Footpaths eroded into grass cover on downstream slopes of left (south) and right (north) non-overflow sections. Two (2) animal burrow holes located on downstream slope of right (north) non-overflow section.	
VERTICAL AND HORIZONTAL ALIGNMENT OF THE CREST	No significant vertical or horizontal misalignment noted.	
RIPRAP FAILURES	N/A	

*REFER TO REPORT SECTIONS 3 AND 7

NON-OVERFLOW SECTION SLOPES

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SETTLEMENT	Downstream slope surface of right non-overflow section is "pitted" in appearance. (Attributed to settlement of soil and rock backfill and washing of fines by infiltration of surface drainage.)	
JUNCTION OF DAM AND ABUTMENTS	Abutment junctions vegetated with grass and appear stable.	
ANY NOTICEABLE SEEPAGE	None	
STAFF GAGE AND RECORDER	None	
DRAINS	None	

GATED SPILLWAY

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CONCRETE SILL	N/A	
APPROACH CHANNEL	N/A	
DISCHARGE CHANNEL	N/A	
BRIDGE AND PIERS	N/A	
GATES AND OPERATION EQUIPMENT	N/A	

UNGATED OVERFLOW SECTION

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CONCRETE WEIR	Overflow section (ogee in shape) constructed of stone blocks.	
APPROACH CHANNEL	N/A	
DISCHARGE CHANNEL	N/A	
BRIDGE AND PIERS	Single lane paved bridge located 250 ft. downstream of dam.	

OUTLET WORKS

(Pond Drain)

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CRACKING AND SPALLING OF CONCRETE SURFACES IN OUTLET CONDUIT	See "Construction Joints". No spalling or cracking of stone block surfaces evident.	
INTAKE STRUCTURE	Slide gates inoperable.	
OUTLET STRUCTURE	Round arch conduit outlet in good condition.	
OUTLET CHANNEL	Right (north) channel bank eroded. Erosion extends from overflow section end wall to single lane bridge located 250 ft. downstream of dam.	
EMERGENCY GATE	None	

RESERVOIR

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SLOPES	Reservoir slopes are primarily covered with trees and vegetation and appear stable. Slopes have gentle to moderate inclinations.	
SEDIMENTATION	Reservoir water appears silt laden. Inlet to Lake Roland shows extensive deposits of sedimentation. Past reports indicate 21,000 cubic yards of material is yearly deposited in Lake Roland reservoir.	

INSTRUMENTATION

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
MONUMENTATION/SURVEYS	N/A	
OBSERVATION WELLS	N/A	
WEIRS	N/A	
PIEZOMETERS	N/A	
OTHER	N/A	

DOWNSTREAM CHANNEL

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CONDITION (OBSTRUCTIONS, DEBRIS, ETC.)	Downstream channel cobble lined, about 60 ft. in width, and stable. No flow obstructions observed.	
SLOPES	Stream channel slopes vegetated with grass and have adequate erosion protection. Stream channel lined with concrete, approximately 2,000 ft. downstream of dam.	
APPROXIMATE NO. OF HOMES AND POPULATION	Lake Roland Dam is located 0.45 mi. upstream from the city limits of Baltimore, Maryland. Substantial property damage and loss of life is expected to occur in the Jones Falls floodplain in the event of a dam failure. The Jones Falls floodplain includes sections of the following communities: Bare Hills, Village of Cross Keys, Woodberry, Hampden, and Baltimore City.	

APPENDIX B

CHECK LIST
ENGINEERING DATA
DESIGN, CONSTRUCTION, OPERATION
PHASE 1

CHECK LIST
ENGINEERING DATA
DESIGN, CONSTRUCTION, OPERATION
PHASE 1

NAME OF DAM Lake Roland Dam

ID # MD 104

ITEM	REMARKS
AS-BUILT DRAWINGS	As-built drawings available from Baltimore City, Department of Public Works, Water Division, Baltimore, Maryland.
REGIONAL VICINITY MAP	See Appendix E. U.S.G.S. 7.5 minute quadrangle map showing dam site location.
CONSTRUCTION HISTORY	Dam designed and built about 1860.
TYPICAL SECTIONS OF DAM	See Drawing Nos. 3 & 4 for cross section view of right (north) non-overflow section and overflow section.
OUTLETS - PLAN DETAILS CONSTRAINTS DISCHARGE RATINGS	Plan view drawing of water supply conduit structure included with as-built drawings. None available.
RAINFALL/RESERVOIR RECORDS	Not available.

ITEM	REMARKS
DESIGN REPORTS	None.
GEOLOGY REPORTS	None.
DESIGN COMPUTATIONS HYDROLOGY & HYDRAULICS DAM STABILITY SEEPAGE STUDIES	None.
MATERIALS INVESTIGATIONS BORING RECORDS LABORATORY FIELD	None.
POST-CONSTRUCTION SURVEYS OF DAM	Post-construction survey of accumulated sediment conducted in 1972 and 1975.
BORROW SOURCES	Unknown.

ITEM	REMARKS
MONITORING SYSTEMS	None.
MODIFICATIONS	Concrete plug installed to seal water supply conduit, April 18, 1958.
HIGH POOL RECORDS	Reportedly, flood stage levels during Agnes reached a maximum stage of about 3 ft. above top of dam.
POST CONSTRUCTION ENGINEERING STUDIES AND REPORTS	None reported.
PRIOR ACCIDENTS OR FAILURE OF DAM DESCRIPTION REPORTS	During Agnes, soil backfill of non-overflow sections was eroded by overtopping flood flows. Downstream side of stone block walls were exposed between El. 231 and El. 210.
MAINTENANCE OPERATION RECORDS	None maintained.

ITEM	REMARKS
OVERFLOW SECTION PLAN	
SECTIONS	See Drawing No. 3 for cross section view of overflow section. (Section interpreted from photographs and as-built drawings.)
DETAILS	
OPERATING EQUIPMENT PLANS & DETAILS	None available.
SPECIFICATIONS	None available.
MISCELLANEOUS	1. Jones Falls Flood Control Study, June 1, 1971, Knoerle, Bender, Stone & Asso., Inc. 2. Dredging of Lake Roland, "Feasibility of Materials Reclamation and Slurry Transport to Cold Spring Quarry", June 1975. 3. The Analysis of the Degradation of Lake Roland, Baltimore, MD, publication No. 6 of the Environmental Studies Program, Goucher College, Towson, MD.

APPENDIX C

HYDROLOGIC AND HYDRAULIC
ENGINEERING DATA
AND CALCULATIONS

LAKE ROLAND
HYDROLOGIC AND HYDRAULIC
ENGINEERING DATA

DRAINAGE AREA CHARACTERISTICS: Predominately residential, some open field, little industrial.

ELEVATION TOP NORMAL POOL (STORAGE CAPACITY): 225 ft. (1,000 ac.-ft., est.)

ELEVATION TOP FLOOD CONTROL POOL (STORAGE CAPACITY): 231 ft. (1,867 ac.-ft., est.)

ELEVATION MAXIMUM DESIGN POOL: 231 ft.

ELEVATION TOP DAM: 231 ft.

OVERFLOW SECTION

a. Elevation 225 ft.
b. Type Ogee weir
c. Width 120 ft.
d. Length N/A
e. Location Spillover Mid-dam
f. Number and Type of Gates None

OUTLET WORKS

a. Type Round arch conduit (constructed of stone blocks)
b. Location Left non-overflow section
c. Entrance Inverts El. 201
d. Exit Inverts El. 200+
e. Emergency Drawdown Facilities None

HYDROMETEOROLOGICAL GAGES

a. Type None
b. Location N/A
c. Records None

MAXIMUM NON-DAMAGING DISCHARGE 5,500 cfs Hurricane Connie, 1955

HEC-1-DAM SAFETY VERSION
HYDROLOGY AND HYDRAULIC ANALYSIS
DATA BASE

NAME OF DAM: Lake Roland Dam — NDI ID No. MD 104

Probable Maximum Precipitation (PMP)	27 in./6 hr.*
Drainage Area	36.76 sq. mi.
Reduction of PMP Rainfall for Data Fit	
Reduce by 16.5%, therefore PMP rainfall =	0.835(27) = 22.5 in.
Adjustments of PMF for Drainage Area	
6 hrs.	89%
12 hrs.	97%
24 hrs.	105%
48 hrs.	117%
Snyder Unit Hydrograph Parameters	
Zone	35**
C_p	0.70
C_t	1.20
L	9.85
L_{ca}	3.92 mi.
$t_p = C_t\,(L \cdot L_{ca})^{0.3} =$	3.60 hrs.
Loss Rates	
Initial Loss	1.00 in.
Constant Loss Rate	0.05 in./hr.
Base Flow Generation Parameters	
Flow at Start of Storm	1.5 cfs/sq. mi. = 55 cfs
Base Flow Cutoff	0.05 x Q peak
Recession Ratio	2.0
Overflow Section Data	
Crest Length	120.0 ft.
Freeboard	6.0 ft.
Discharge Coefficient	3.06
Exponent	1.5
Discharge Capacity	5,400.0 cfs
Breach Parameters	
Section Width	126.0 ft.
Section Height	24.0 ft.
Duration of Failure	0.1 hr.
Depth of Maximum Overtopping Prior to Failure	4.5 ft.

*Hydrometerological Report 33
**Hydrological zone defined by Corps of Engineers, Baltimore District, for determining Snyder's Coefficients (C_p and C_t).

```
A1        ANALYSIS OF DAM OVERTOPPING - LAKE ROLAND
A2         MULTIPLES OF PMF ROUTED THROUGH RESERVOIR
A3             AND DOWNSTREAM
B    :00          5                                              4
B1     5
J      1     8          1
J1    .1    .15      .25      .3      .35     .4      .5       1.
K            1                                          1
K1        CALCULATION OF INFLOW HYDROGRAPH FOR LAKE ROLAND
M      1         1  36.76             36.76                              1
P           22.5       89       97      105
T                                                        1.      .05
W   3.6        .7
X  -1.5      -.05       2.
K      1        2                                        1
K1        ROUTED FLOWS THROUGH LAKE ROLAND
Y                                  1         1
Y1     1
$S          1000.   2300.   3400.    5400.    8300.   1000.
$E  200.    225.     234.    240.     250.     260.   12650.
$S  255.    120.    3.06     1.5                       275.
$D  231.    3.1      1.5    126.
$L  202.   223.5   255.5    288.     320.3    352.5   384.8    406.3
$V  231.    233.    236.    239.     242.     245.    248.     250.
K      1        3                                        1
K1        MOD PULS ROUTING FROM DAM TO SECTION THREE
Y                                  1         1
Y1     1
Y6   .07      .03      .07    190.     300.    2000.    .001
Y7          300.     500.    240.    1300.     200.   1370.    190.    1430.   190.
Y7  1500.   200.    2200.    240.    3000.     300.
K      1        4                                        1
K1        MOD PULS ROUTING FROM STA 3 TO STA 4
Y                                  1         1
Y1     1
Y5   .07      .03      .07    188.     300.    2000.    .001
Y7          300.    1000.    240.    1800.     200.   1870.    188.    1930.   188.
Y7  2000.   200.    2300.    260.    2500.     300.
K     99
```

PEAK FLOW AND STORAGE (END OF PERIOD) SUMMARY FOR MULTIPLE PLAN-RATIO ECONOMIC COMPUTATIONS
FLOWS IN CUBIC FEET PER SECOND (CUBIC METERS PER SECOND)
AREA IN SQUARE MILES (SQUARE KILOMETERS)

RATIOS APPLIED TO FLOWS

OPERATION	STATION	AREA	PLAN	RATIO 1 .10	RATIO 2 .15	RATIO 3 .25	RATIO 4 .30	RATIO 5 .35	RATIO 6 .40	RATIO 7 .50	RATIO 8 1.00
HYDROGRAPH AT	1 (	36.76 95.21)	1 (	6155. 174.29)(	9232. 261.44)(	15387. 435.73)(	18465. 522.87)(	21542. 610.02)(	24620. 697.16)(	30775. 871.45)(	61550. 1742.90)(
ROUTED TO	2 (	36.76 95.21)	1 (	5332. 150.98)(	8685. 245.94)(	14886. 421.51)(	17916. 507.32)(	20975. 593.94)(	24047. 680.93)(	30195. 855.02)(	60836. 1722.68)(
ROUTED TO	3 (	36.76 95.21)	1 (	5329. 150.91)(	8679. 245.76)(	14880. 421.34)(	17907. 507.06)(	20965. 593.68)(	24037. 680.66)(	30178. 854.53)(	60810. 1721.95)(
ROUTED TO	4 (	36.76 95.21)	1 (	5327. 150.84)(	8672. 245.57)(	14873. 421.16)(	17899. 506.83)(	20960. 593.52)(	24027. 680.37)(	30171. 854.34)(	60795. 1721.53)(

COMPUTER INPUT: OVERTOPPING ANALYSIS

SUMMARY OF DAM SAFETY ANALYSIS

PLAN 1 ....

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	225.00	225.00	231.00
STORAGE	1000.	1000.	1867.
OUTFLOW	0.	0.	5397.

RATIO OF PMF	MAXIMUM RESERVOIR W.S.ELEV	MAXIMUM DEPTH OVER DAM	MAXIMUM STORAGE AC-FT	MAXIMUM OUTFLOW CFS	DURATION OVER TOP HOURS	TIME OF MAX OUTFLOW HOURS	TIME OF FAILURE HOURS
.10	230.95	0.00	1860.	5332.	0.00	20.08	0.00
.15	232.45	1.49	2081.	8685.	4.33	19.67	0.00
.25	234.43	3.43	2379.	14886.	6.83	19.50	0.00
.30	235.23	4.23	2525.	17916.	7.50	19.50	0.00
.35	235.97	4.97	2661.	20975.	8.00	19.50	0.00
.40	236.67	5.67	2789.	24047.	8.42	19.42	0.00
.50	237.95	6.95	3024.	30195.	9.17	19.42	0.00
1.00	243.04	12.04	4008.	60836.	10.58	19.33	0.00

PLAN 1 STATION 3

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
.10	5329.	198.0	20.17
.15	8679.	200.5	19.75
.25	14880.	203.3	19.58
.30	17907.	204.4	19.58
.35	20965.	205.5	19.50
.40	24037.	206.6	19.50
.50	30178.	208.3	19.42
1.00	60810.	215.0	19.33

PLAN 1 STATION 4

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
.10	5327.	196.5	20.25
.15	8672.	199.3	19.83
.25	14873.	202.0	19.67
.30	17899.	203.2	19.58
.35	20960.	204.4	19.58
.40	24027.	205.5	19.58
.50	30171.	207.2	19.50
1.00	60795.	214.1	19.42

SUMMARY OF OVERTOPPING ANALYSIS AND FLOOD ROUTING

```
A1              ANALYSIS OF EFFECTS OF STRUCTURAL FAILURE OF DAM
A2               HYDROLOGIC-HYDRAULIC CONSEQUENCES DOWNSTREAM FOR VARIOUS
A3              MODES OF FAILURE OF LAKE ROLAND DAM
B      300                      5                                                  4
B1       5
J        5           1          1
J1     .35
K                    1                                         1
K1              CALCULATION OF INFLOW HYDROGRAPH FOR LAKE ROLAND
M        1           1  36.76                 36.76                                1
P                22.5         89        97      105
T                                                              1.       .05
W      3.6         .7
X     -1.5       -.05         2.
K        1           2                                         1
K1              ROUTED FLOWS THROUGH LAKE ROLAND
Y                                         1       1
Y1       1                                                  1000.
$S              1000.   2300.   3400.   5400.   8300.   12650.
$E     200.      225.    234.    240.    250.    260.     275.
$$     225.      120.   3.06     1.5
$D     231.      3.1     1.5    126.
$L     242.    223.5   255.8    288.   320.3   352.5    384.8    406.3
$V     231.     233.    236.    239.    242.    245.     248.     250.
$B     126.             231.      .1    225.   235.5
$B     126.           229.5       .1    225.   235.5
$B     126.             225.      .1    225.   235.5
$B     126.             207.      .1    225.   235.5
$H              2.625   207.      .1    225.   235.5
K        1           3                                         1
K1              MOD PULS ROUTING FROM DAM TO SECTION THREE
Y                                         1       1
Y1       1
Y6     .07       .03     .07    190.    300.   2000.     .001
Y7               300.    900.   240.   1300.    200.    1370.     190.     1430.   190.
Y7    1500.      200.   2200.   240.   3000.    300.
K        1           4                                         1
K1              MOD PULS ROUTING FROM STA 3 TO STA 4
Y                                         1       1
Y1       1
Y6     .07       .03     .07    188.    300.   2000.     .001
Y7               300.   1000.   240.   1800.    200.    1870.     188.     1930.   188.
Y7    2000.      200.   2300.   260.   2500.    300.
K       99
A
A
A
A
A
```

COMPUTER INPUT: BREACH ANALYSIS

```
                                              DAM BREACH DATA
                          BRWID       Z       ELBM      TFAIL     WSEL     FAILEL
PLAN 1                     126.      0.00    231.00       .10    225.00    235.50
BEGIN DAM FAILURE AT 18.50 HOURS
PEAK OUTFLOW IS    20973. AT TIME  19.50 HOURS

                                              DAM BREACH DATA
                          BRWID       Z       ELBM      TFAIL     WSEL     FAILEL
PLAN 2                     126.      0.00    229.50       .10    225.00    235.50
BEGIN DAM FAILURE AT 18.50 HOURS
PEAK OUTFLOW IS    21406. AT TIME  19.25 HOURS

                                              DAM BREACH DATA
                          BRWID       Z       ELBM      TFAIL     WSEL     FAILEL
PLAN 3                     126.      0.00    225.00       .10    225.00    235.50
BEGIN DAM FAILURE AT 18.50 HOURS
PEAK OUTFLOW IS    28051. AT TIME  18.60 HOURS

                                              DAM BREACH DATA
                          BRWID       Z       ELBM      TFAIL     WSEL     FAILEL
PLAN 4                     126.      0.00    207.00       .10    225.00    235.50
BEGIN DAM FAILURE AT 18.50 HOURS
PEAK OUTFLOW IS    68903. AT TIME  18.60 HOURS

                                              DAM BREACH DATA
                          BRWID       Z       ELBM      TFAIL     WSEL     FAILEL
PLAN 5                       0.      2.63    207.00       .10    225.00    235.50
BEGIN DAM FAILURE AT 18.50 HOURS
PEAK OUTFLOW IS    41820. AT TIME  18.60 HOURS
```

COMPUTER INPUT: BREACH ANALYSIS

SUMMARY OF DAM SAFETY ANALYSIS

PLAN 1 ....

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	225.00	225.00	231.00
STORAGE	1000.	1000.	1867.
OUTFLOW	0.	0.	5397.

RATIO OF PMF	MAXIMUM RESERVOIR W.S.ELEV	MAXIMUM DEPTH OVER DAM	MAXIMUM STORAGE AC-FT	MAXIMUM OUTFLOW CFS	DURATION OVER TOP HOURS	TIME OF MAX OUTFLOW HOURS	TIME OF FAILURE HOURS
.35	235.97	4.97	2662.	20973.	8.00	19.50	18.50

PLAN 2 ....

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	225.00	225.00	231.00
STORAGE	1000.	1000.	1867.
OUTFLOW	0.	0.	5397.

RATIO OF PMF	MAXIMUM RESERVOIR W.S.ELEV	MAXIMUM DEPTH OVER DAM	MAXIMUM STORAGE AC-FT	MAXIMUM OUTFLOW CFS	DURATION OVER TOP HOURS	TIME OF MAX OUTFLOW HOURS	TIME OF FAILURE HOURS
.35	235.60	4.60	2593.	21406.	7.67	19.25	18.50

PLAN 3 ....

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	225.00	225.00	231.00
STORAGE	1000.	1000.	1867.
OUTFLOW	0.	0.	5397.

RATIO OF PMF	MAXIMUM RESERVOIR W.S.ELEV	MAXIMUM DEPTH OVER DAM	MAXIMUM STORAGE AC-FT	MAXIMUM OUTFLOW CFS	DURATION OVER TOP HOURS	TIME OF MAX OUTFLOW HOURS	TIME OF FAILURE HOURS
.35	235.52	4.52	2579.	28051.	6.17	18.60	18.50

PLAN 4 ....

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	225.00	225.00	231.00
STORAGE	1000.	1000.	1867.
OUTFLOW	0.	0.	5397.

RATIO OF PMF	MAXIMUM RESERVOIR W.S.ELEV	MAXIMUM DEPTH OVER DAM	MAXIMUM STORAGE AC-FT	MAXIMUM OUTFLOW CFS	DURATION OVER TOP HOURS	TIME OF MAX OUTFLOW HOURS	TIME OF FAILURE HOURS
.35	235.51	4.51	2577.	68903.	2.58	18.60	18.50

PLAN 5 ....

	INITIAL VALUF	SPILLWAY CREST	TOP OF DAM
ELEVATION	225.00	225.00	231.00
STORAGE	1000.	1000.	1867.
OUTFLOW	0.	0.	5397.

RATIO OF PMF	MAXIMUM RESERVOIR W.S.ELEV	MAXIMUM DEPTH OVER DAM	MAXIMUM STORAGE AC-FT	MAXIMUM OUTFLOW CFS	DURATION OVER TOP HOURS	TIME OF MAX OUTFLOW HOURS	TIME OF FAILURE HOURS
.35	235.53	4.53	2580.	41820.	3.42	18.60	18.50

COMPUTER OUTPUT: BREACH ANALYSIS

PLAN 1 STATION 3

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
.35	20963.	205.5	19.50

PLAN 2 STATION 3

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
.35	21402.	205.6	19.25

PLAN 3 STATION 3

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
.35	26516.	207.4	18.75

PLAN 4 STATION 3

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
.35	56413.	214.2	18.75

PLAN 5 STATION 3

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
.35	36806.	209.9	18.75

DAMAGE STATION #3: BREACH ANALYSIS

PLAN 1 STATION 4

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
.35	20958.	204.4	19.58

PLAN 2 STATION 4

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
.35	21400.	204.5	19.33

PLAN 3 STATION 4

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
.35	26159.	206.1	18.83

PLAN 4 STATION 4

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
.35	53678.	212.8	18.83

PLAN 5 STATION 4

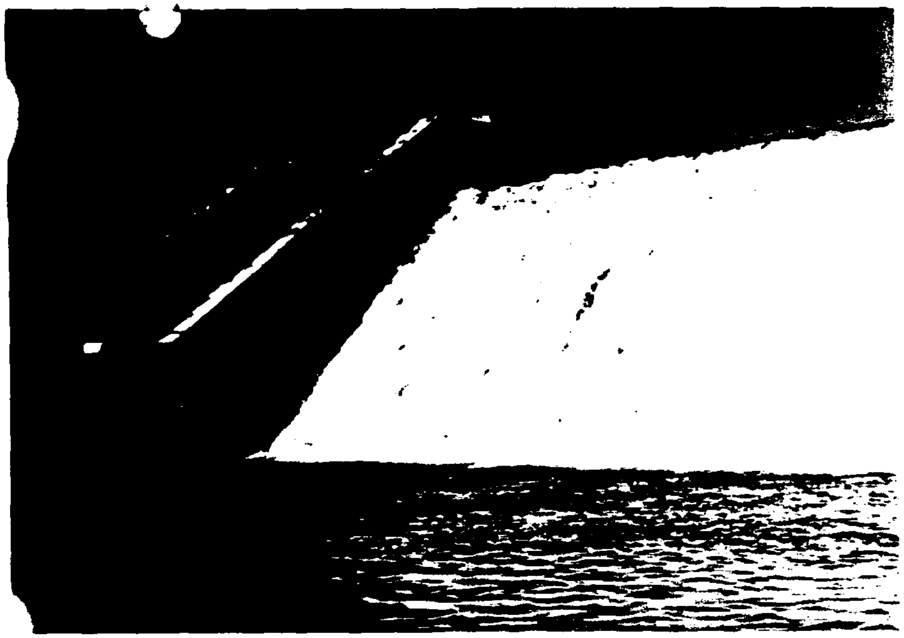
RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
.35	35654.	208.6	18.83

DAMAGE STATION #4: BREACH ANALYSIS

APPENDIX D

PHOTOGRAPHS

PHOTOGRAPH 1	View of overflow section and right (north) overflow section sidewall.
PHOTOGRAPH 2	View of overflow section and left (south) overflow section sidewall.
PHOTOGRAPH 3	Upstream view of right (north) non-overflow section. Note tree growing on upstream slope.
PHOTOGRAPH 4	Downstream view of right (north) non-overflow section. Note eroded footpath and pitted surface.



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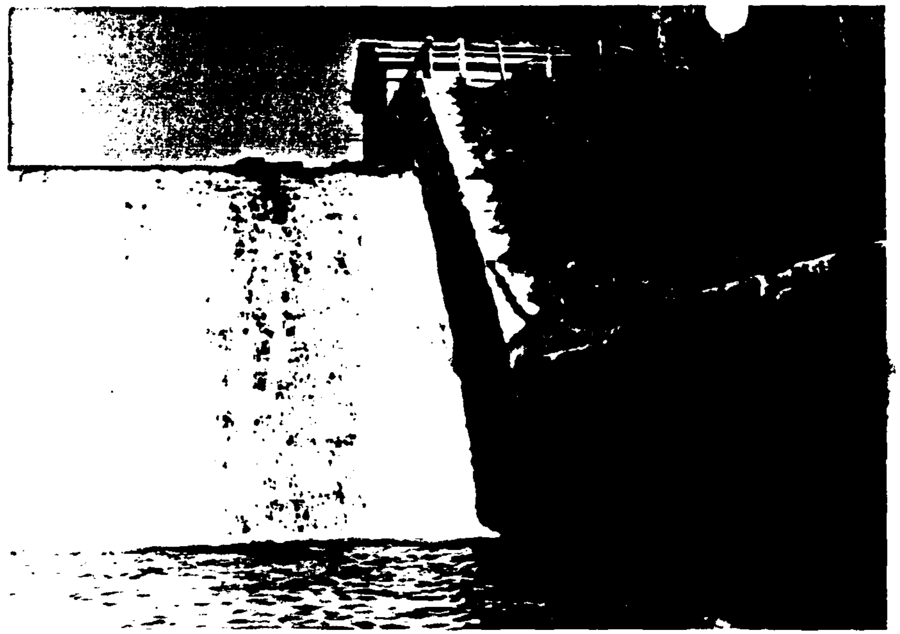

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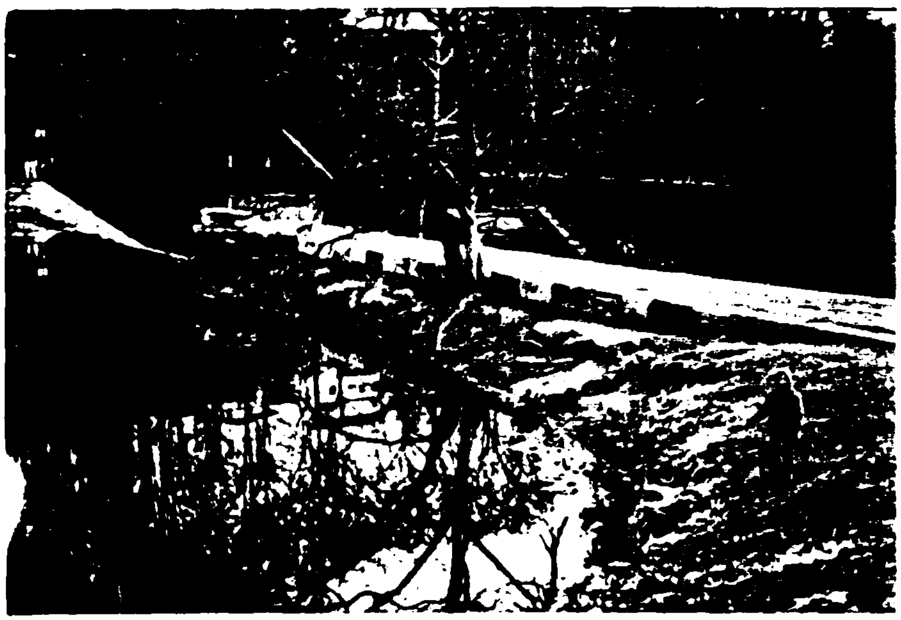

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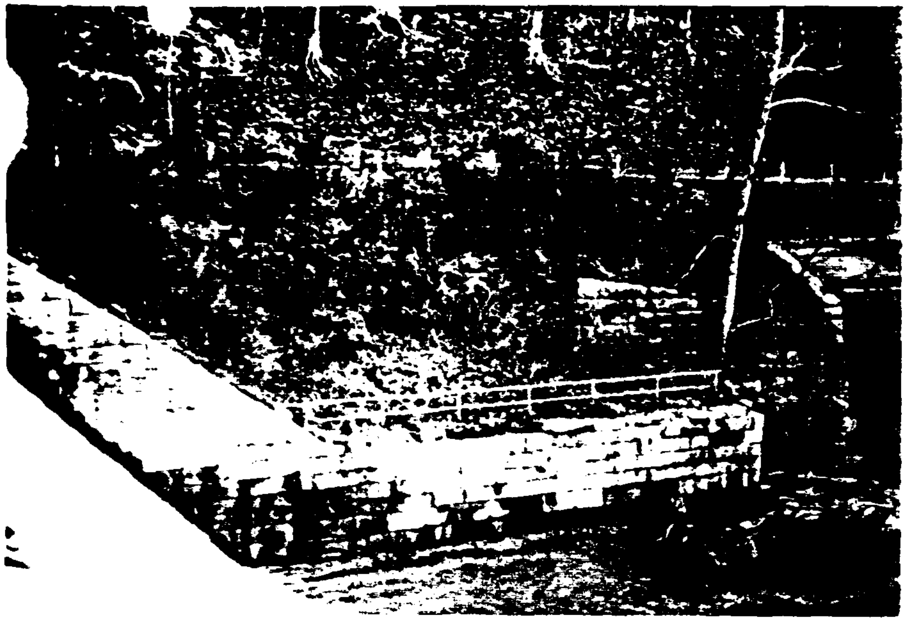
PHOTOGRAPH 5	View of seepage emanating from left (south) overflow section sidewall and missing capping stones.
PHOTOGRAPH 6	Gate house located on left (south) abutment.
PHOTOGRAPH 7	Outlet of round arch conduit reservoir drain.
PHOTOGRAPH 8	Bridge overpass 250 ft. downstream of dam.



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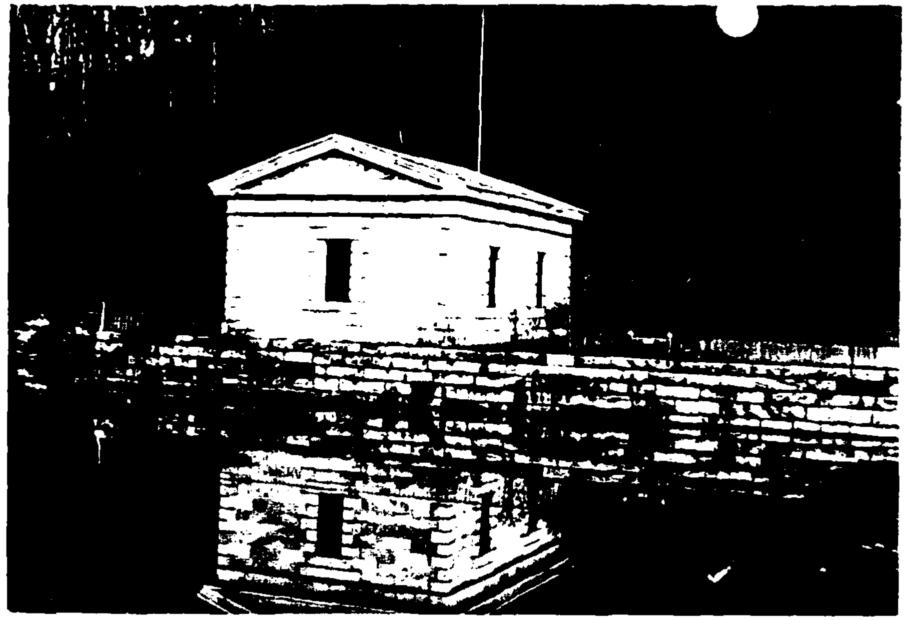

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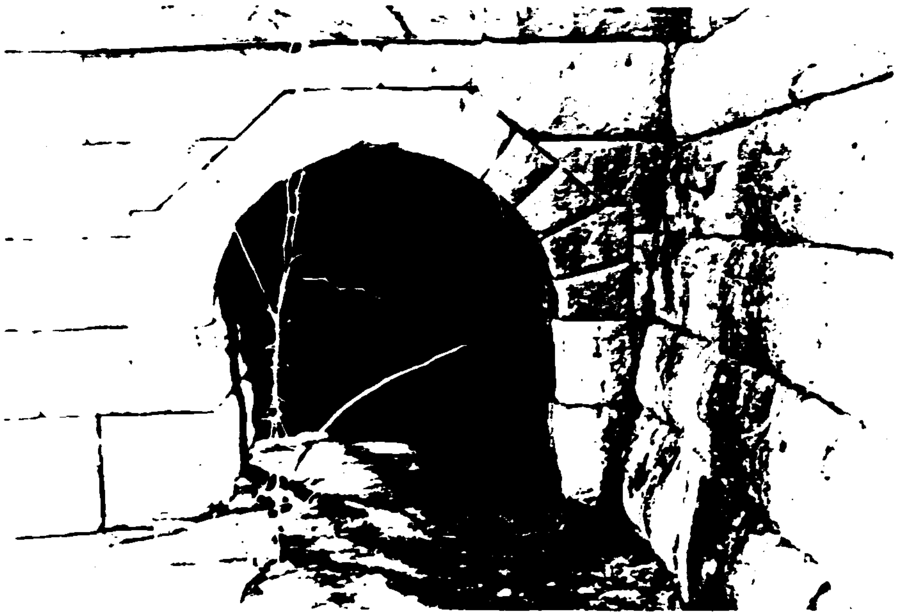

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PHOTOGRAPH 9 — Erosion of right (north) non-overflow section slope by Hurricane Agnes (1972).

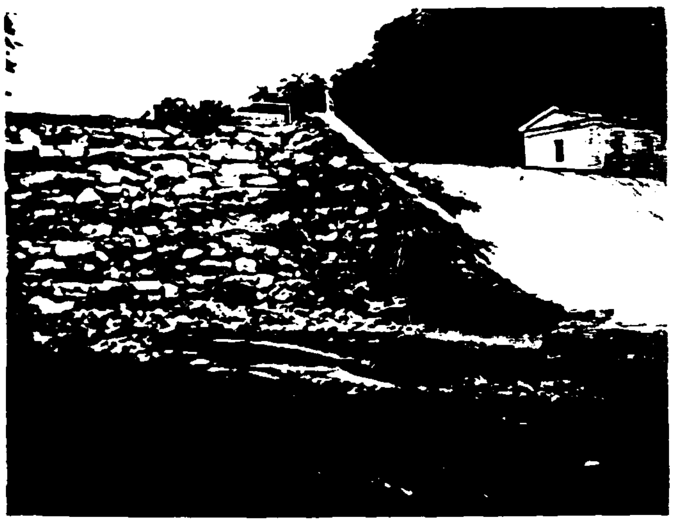
PHOTOGRAPH 10 — Erosion of left (south) non-overflow section slope by Hurricane Agnes (1972).

PHOTOGRAPH 11 — Erosion of right (north) non-overflow section slope by Hurricane Agnes (1972).

PHOTOGRAPH 12 — Erosion of left (south) non-overflow section slope by Hurricane Agnes (1972).

Lake Roland Dam
9

Lake Roland Dam
10



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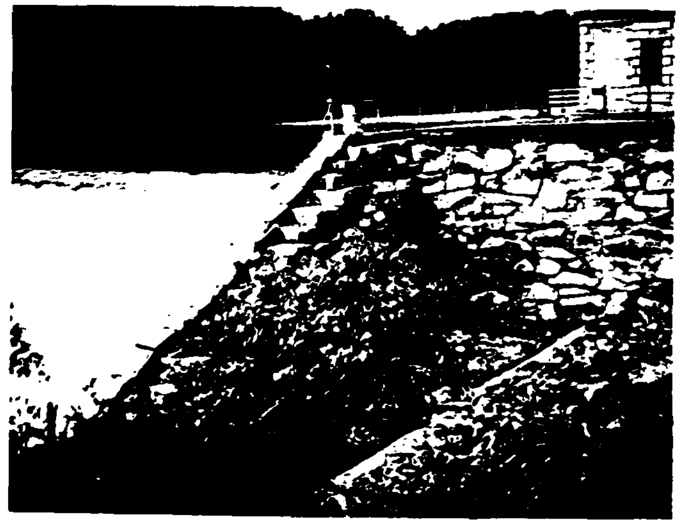

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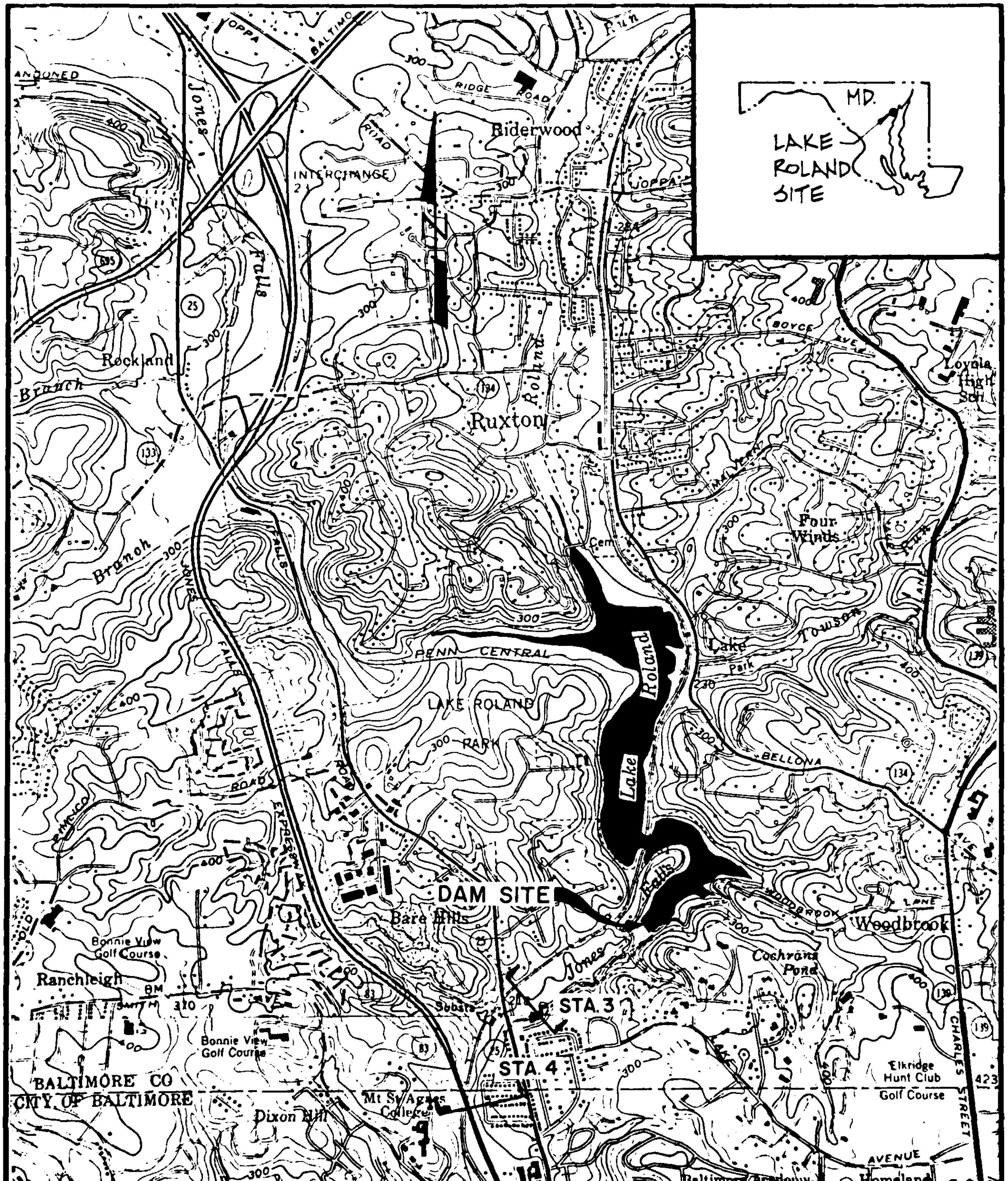
APPENDIX E

REGIONAL LOCATION PLAN



DATE: JULY 19, 1979	NATIONAL DAM INSPECTION PROGRAM	LOCATION PLAN LAKE ROLAND DAM SITE
SCALE: 1:24000		
DR: JLM CK: TED	ACKENHEIL & ASSOCIATES CONSULTING ENGINEERS BALTIMORE, MD.	
DWG. NO. E1		

APPENDIX F

REGIONAL GEOLOGY

LAKE ROLAND DAM
NDI I.D. NO. MD 104
REGIONAL GEOLOGY

Lake Roland Dam is located in the Coastal Plain Physiographic Province of Baltimore County. The predominate geologic structures of this region are the Towson Dome, Chattolanee Dome, and Laurel Belt.

The dam structure is situated on the western edge of the Towson Dome within the Baltimore Gneiss formation. The Towson Dome consists predominately of dark and light biotite-microline-quartz-plagioclase gneiss. Foliation of this layered gneiss member strikes N 70^{o}-72^{o} W and is inclined about 76^{o} - 90^{o}. Lake Roland Dam is located about 0.2 miles east of the Ruxton Thrust Fault and 0.7 miles southwest of a minor thrust fault bordering the Laurel Belt.

The lithologic unit of the Laurel Belt structure is the Mount Washington Amphibolite, which consists of a fine to medium grained amphibolite locally occurring with pyroxene.

References

Maryland Geological Survey, 1976, Geologic Map of Baltimore County and City.

Maryland Geological Survey, 1929, Baltimore County.

LAKE ROLAND DAM
NDI I.D. NO. MD 104
REGIONAL GEOLOGY

Lake Roland Dam is located in the Coastal Plain Physiographic Province of Baltimore County. The predominant geologic structures of this region are the Towson Dome, Chattolanee Dome, and Laurel Belt.

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The lithologic unit of the Laurel Belt structure is the Mount Washington Amphibolite, which consists of a fine to medium grained amphibolite locally occurring with pyroxene.

References

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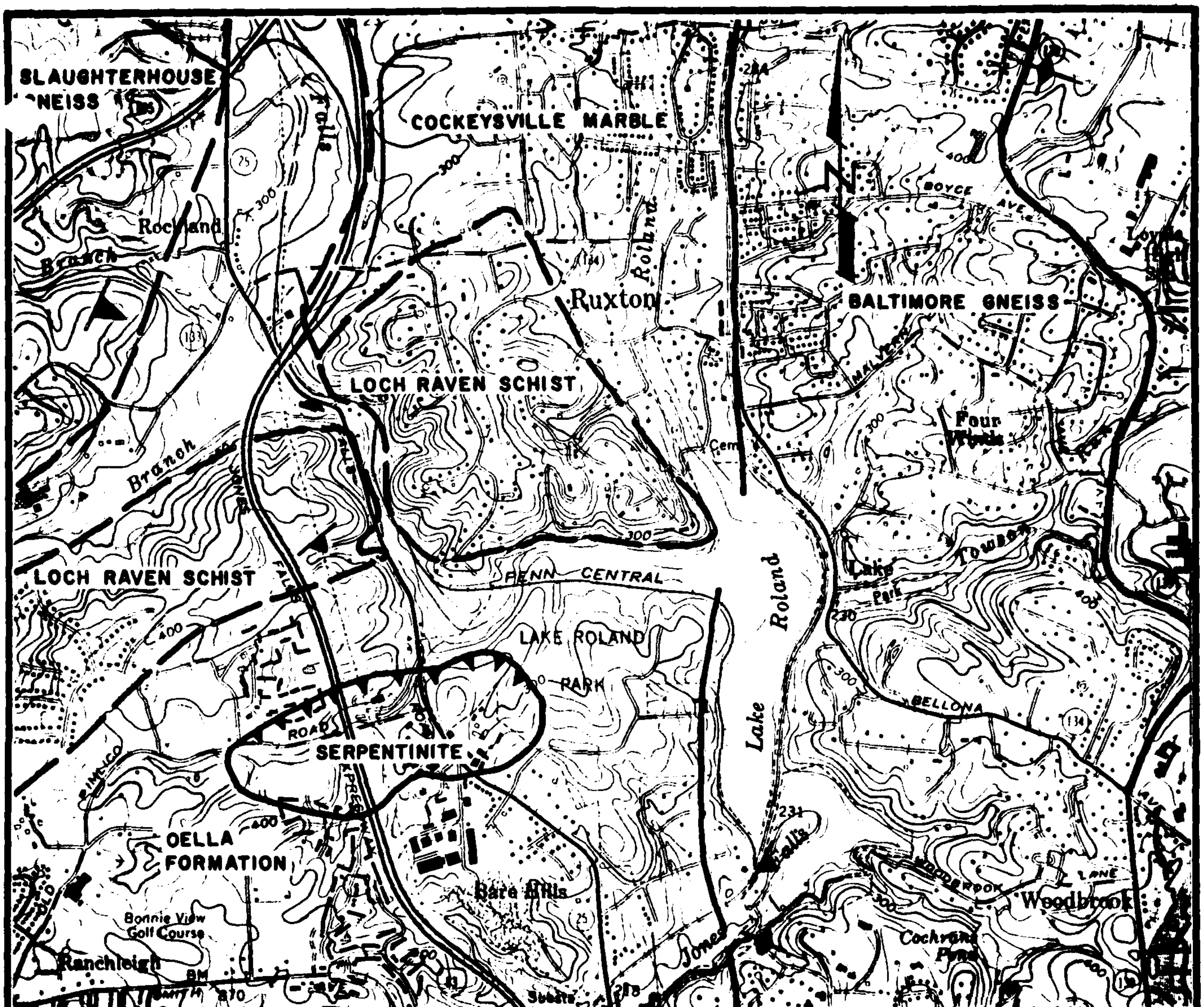
Maryland Geological Survey, 1929, Baltimore County.



COCKEYSVILLE QUADRANGLE, BALTIMORE COUNTY, MARYLAND

SCALE: 0 — 1/2 MILE 1:24000

CONTOUR INTERVAL 20FT. DATUM IS MEAN SEA LEVEL

— — — FORMATION CONTACT

——— NORMAL FAULT AND FORMATION CONTACT

▼▼▼▼ THRUST FAULT TEETH ON UPPER PLATE

FOLIATION

1°- 25° 51°- 75° 76°- 90°

DATA OBTAINED FROM MARYLAND GEOLOGICAL SURVEY'S GEOLOGIC MAP OF BALTIMORE COUNTY AND CITY, 1976

DATE: JULY 13, 1979	NATIONAL DAM INSPECTION PROGRAM	SITE GEOLOGY OF LAKE ROLAND DAM
SCALE: AS SHOWN		
DR: JLM / CK: TED	ACKENHEIL & ASSOCIATES CONSULTING ENGINEERS BALTIMORE, MD.	
DWG. NO. F2		

APPENDIX G

STABILITY CALCULATIONS

PAD
7/8/79
TED
7/16/79

SHEET NO. G-1 OF

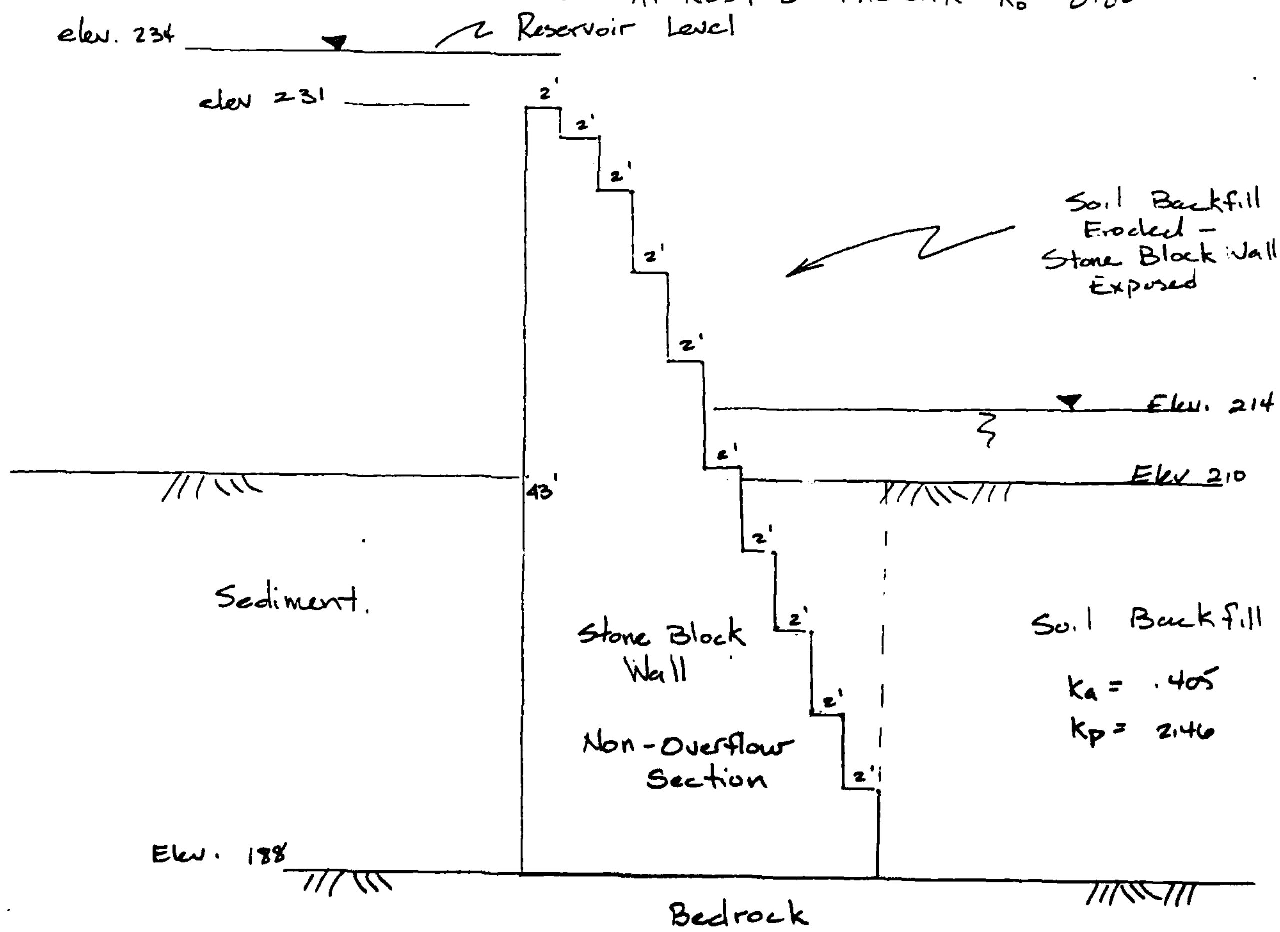
Analysis #1 - Stability of Lake Roland Dam
During Conditions of Hurricane Agnes

Conditions

1. Dam overtopped by 3 ft. (Elev. 234)
2. Backfill of overflow section eroded to el. 210

Assumptions

1. Non-Overflow section will fail before overflow section
2. Unit wt. of stone blocks = 168 pcf
3. Soil backfill - γ = 122.4 pcf, c = 200 psf, ϕ = 25°
4. C_f = 0.65
5. Depth of Reservoir = 18 ft. max.
6. Sediment - S.G. = 1.45, c = 0, ϕ = 0
7. level of tailwater = El. 214
8. At Rest coefficient K_o = 0.80



Scale: 1" = 10'

DFD
7/8/79
TED
7/16/79

Analysis #1 - Stability of Lake Roland Dam
During Conditions of Hurricane Agnes SHEET NO S-2 OF

Wt. of Stone Block Wall / ft.

Wt. = 2 (43 + 41 + 38 + 34 + 29 + 23 + 18 + 14 + 9 + 5) × 168

W_w = Wt. = 85,344 lbs = 85.3 K/ft.

Active Condition - Upstream Side

el. 231 p_a = 3 × .0624 = .187 ksf

el. 210 p_a = 24 × .0624 = 1.498 ksf

el. 188 p_a = (46 × .0624) + (1.45 − 1) .0624 × 22 = 3.488 ksf.

Passive Condition - Downstream Side

el. 210 ↑ p_p = 4 × .0624 = .250 ksf

el. 210 ↓ p_p = 0 + 2(.200) $\sqrt{2.46}$ + 4 × .0624 = .877 ksf

el. 188 ↑ p_p = 22 × .0600 × 2.46 + 2(.200) $\sqrt{2.46}$ + 26 × .0624
= 5.497 ksf

At. Rest Condition - Downstream Side

el. 210 ↑ p_o = .250 ksf

el. 188 ↑ - p_o = (22 × .060 × .8) + (26 × .0624) = 2.68 ksf

Uplift Force

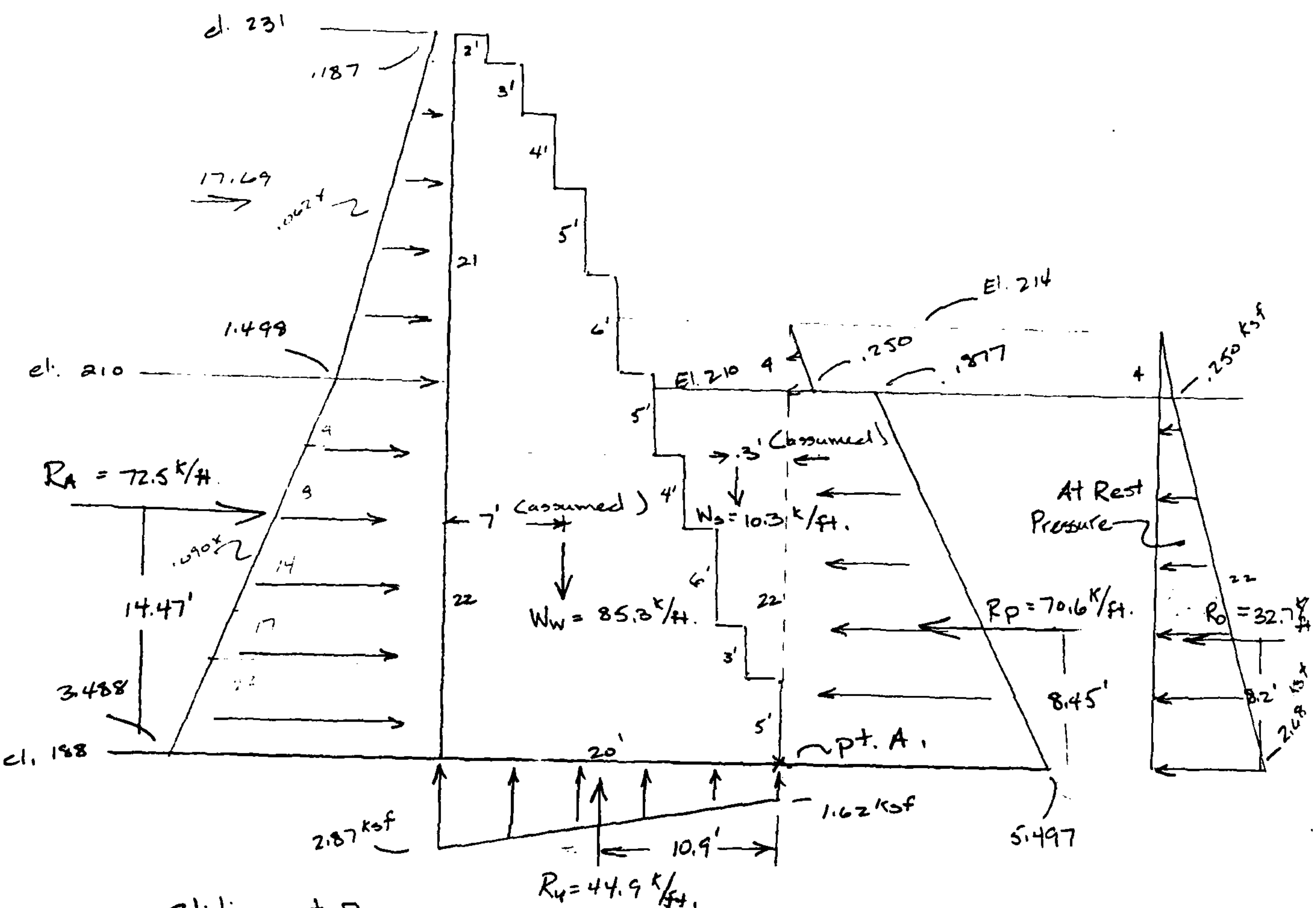
Assume there are no drainage provisions for uplift reduction.

@ heel H_2 = headwater = 46 × .0624 = 2.87 ksf

@ Toe H_1 = Tailwater = 26 × .0624 = 1.62 ksf

Analysis #1 - Stability of Lake Roland Dam During Conditions of Hurricane Agnes.

Forces on Dam



Sliding at Base

Acting Force = 72.5 k/ft.

Passive - Resisting Force = (85.3 + 10.3 - 44.9) .65 + 70.6 = 103.5 k/ft.

$F.S. = \frac{103.5}{72.5} = 1.43$

At Rest - Resisting Force = (85.3 + 10.3 - 44.9) .65 + 32.7 = 65.7 k/ft.

$F.S. = \frac{65.7}{72.5} = 0.90$

Overturning.

ΣM_a ↻ = 72.5 × 14.47 + 44.9 × 10.9 = 1539 ′k/′

passive ΣM_a ↺ = 85.3 × 14 + 10.3 × 3 + 70.6 × 8.45 = 1822 ′k/′

at rest ΣM_a ↺ = 85.3 × 14 + 10.3 × 3 + 32.7 × 8.2 = 1493 ′k/′

$F.S._{passive} = \frac{1822}{1539} = 1.18$ $\quad F.S._{at\ rest} = \frac{1493}{1539} = 0.97$

PAD
7/8/79
TED
7/16/79

Analysis #2 - Stability of Lake Roland Dam
Existing Conditions.

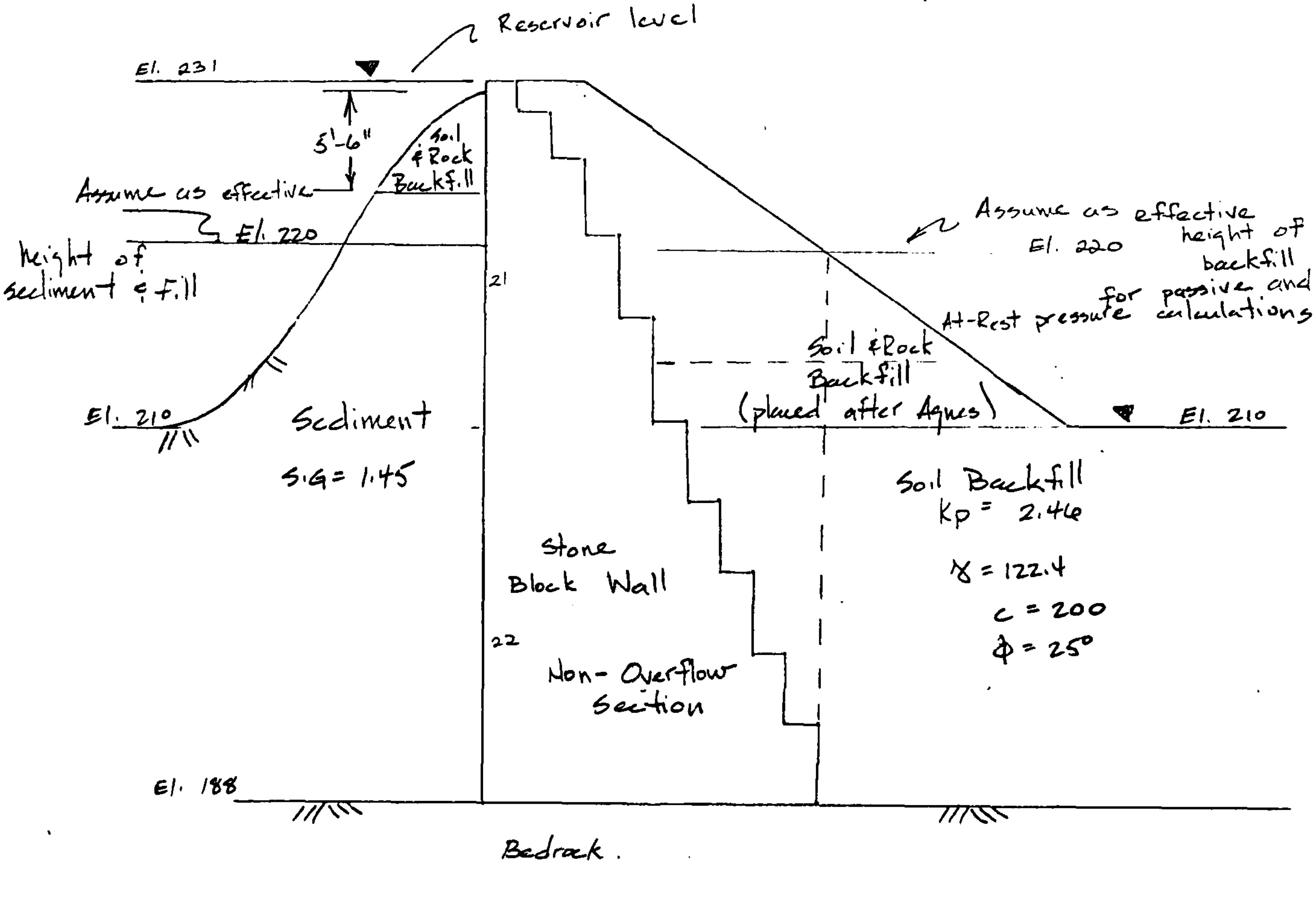
SHEET NO G-4 OF ____

Conditions

1. Reservoir level @ top of dam.
2. Complete soil cover downstream of Stone Block Wall

Assumptions

- See page G-1
- Assume downstream Backfill Conditions are homogeneous.



Scale: 1" = 10'

PAD 7/8/79
TED 7/16/79

Analysis #2 - Stability of Lake Roland Dam
Existing Conditions

SHEET NO. G-5 OF

Wt. of Stone Block Wall / ft.

W_W = 85.3 K/ft. → see pg. G-2

Active Condition - Upstream Side

el. 220 p_a = 11 × .0624 = 0.69 ksf
el. 188 p_a = 32 × .0624 (1.45-1) + 43 × .0624 = 3.58 ksf

Passive Condition - Downstream Side

El. 220 p_p = 0 + 2 × .200 $\sqrt{2.46}$ + 0 = .627 ksf
El. 210 p_p = 10 × .1224 × 2.46 + 2 × .2 $\sqrt{2.46}$ + 0 = 3.64 ksf
El. 188 p_p = [10 × .1224 + 22 × .06] 2.46 + 2 × .2 $\sqrt{2.46}$
+ 22 × .0624 = 8.25 ksf

At Rest Condition - Downstream Side.

El. 220 p_o = 0
El. 210 p_o = 10 × .1224 × .8 = .98 ksf

El. 188 p_o = (10 × .1224 × .8) + (22 × .06 × .8) + (.0624 × 22)
= 3.41 Ksf

Wt of soil on Stone Block Wall - downstream Side.

W_s = 2(2 + 5 + 8 + 12 + 16 + 20 + 22 + 26 + 28) × .1224
W_s = 34.0 K/ft.

Uplift Force

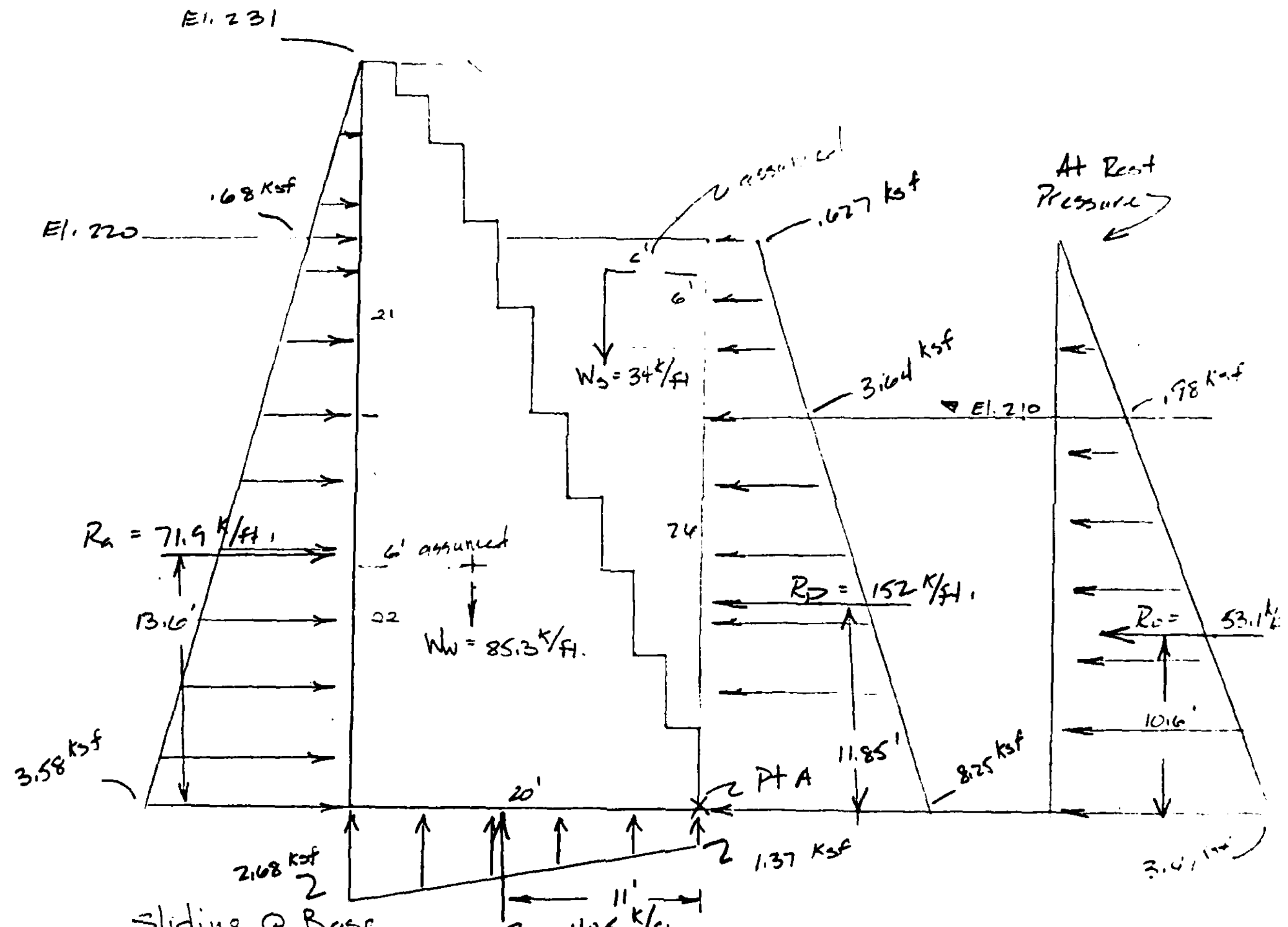
H_2 = 43 × .0624 = 2.68 ksf
H_1 = 22 × .0624 = 1.37 ksf

PAD.
7.8.79
TCD
7/16/79

Analysis #2 - Stability of Lake Roland Dam
Existing Conditions

SHEET NO G-6 OF

Forces on Dam



Sliding @ Base

Acting Force = 71.9 k/ft

Resisting Force (passive) = (85.3 + 34 − 40.5) .65 + 152 = 203.2 k/ft.

Resisting Force (at-rest) = (85.3 + 34 − 40.5) .65 + 53.1 = 104.3 k/ft.

F.S. passive = $\frac{203.2}{71.9}$ = 2.83   F.S. at rest = $\frac{104.3}{71.9}$ = 1.45

Overturning - Pt A

ΣM_A ↻ = 71.9 × 13.6 + 40.5 × 11 = 1423 'K/'

ΣM_A ↺ passive = 85.3 × 14 + 34 × 6 + 152 × 11.85 = 3199   F.S. = 2.3

ΣM_A ↺ at rest = 85.3 × 14 + 34 × 6 + 53.1 × 10.6 = 1961   F.S. = 1.4

PAD
7/8/79
TED
7/6/78

PROJECT NO

Analysis No. 3 - Stability of Lake Roland Dam
- 5 ft. overtopping - Removal of soil cover downstream of Stone Block Wall

Conditions.

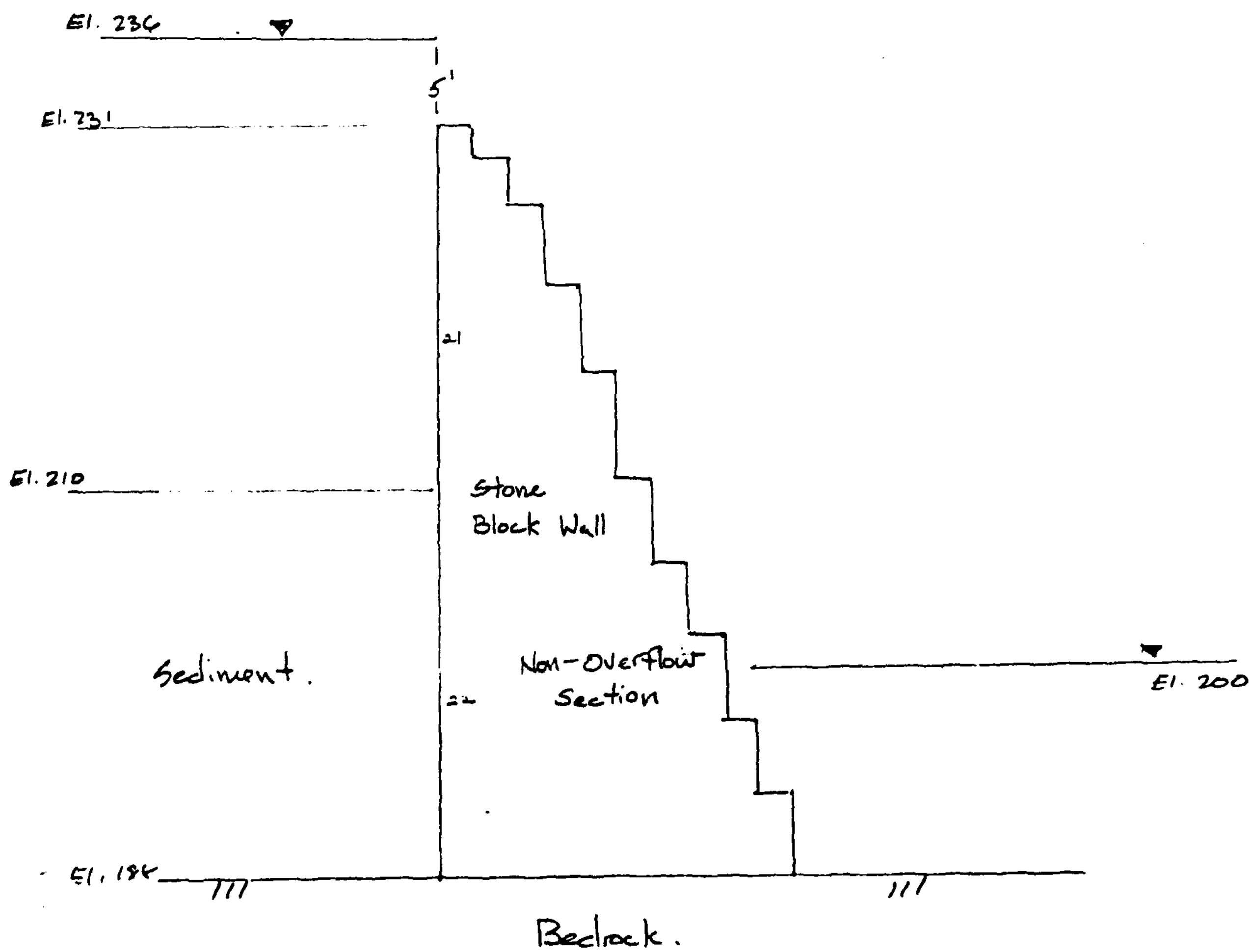
1. Non-Overflow Section overtopped by 5 ft. (El. 236)

2. Backfill of stone Block Wall eroded to bedrock - elev. 188.

Assumptions - see page G-1

exception: level of tailwater = El. 200

5' of overtopping completely erode soil backfill.



FAD
7/8/79
TED
7/16/78

SHEET NO. G-8 OF

Analysis No. 3 Stability of Lake Roland Dam

5 ft. overtopping. Removal of soil cover downstream of stone block wall

Active Pressure.

El. 210 $p_a = 26 \times .0624 = 1.622$ ksf

El. 188 $p_a = 48 \times .0624 + (1.45-1) \times .0624 \times 22 = 3.613$ ksf

El. 231 = $p_a = 5 \times .0624 = 0.312$ ksf

$W_W = 85.3$ k/ft.

$-p_u = .225$ k/ft.

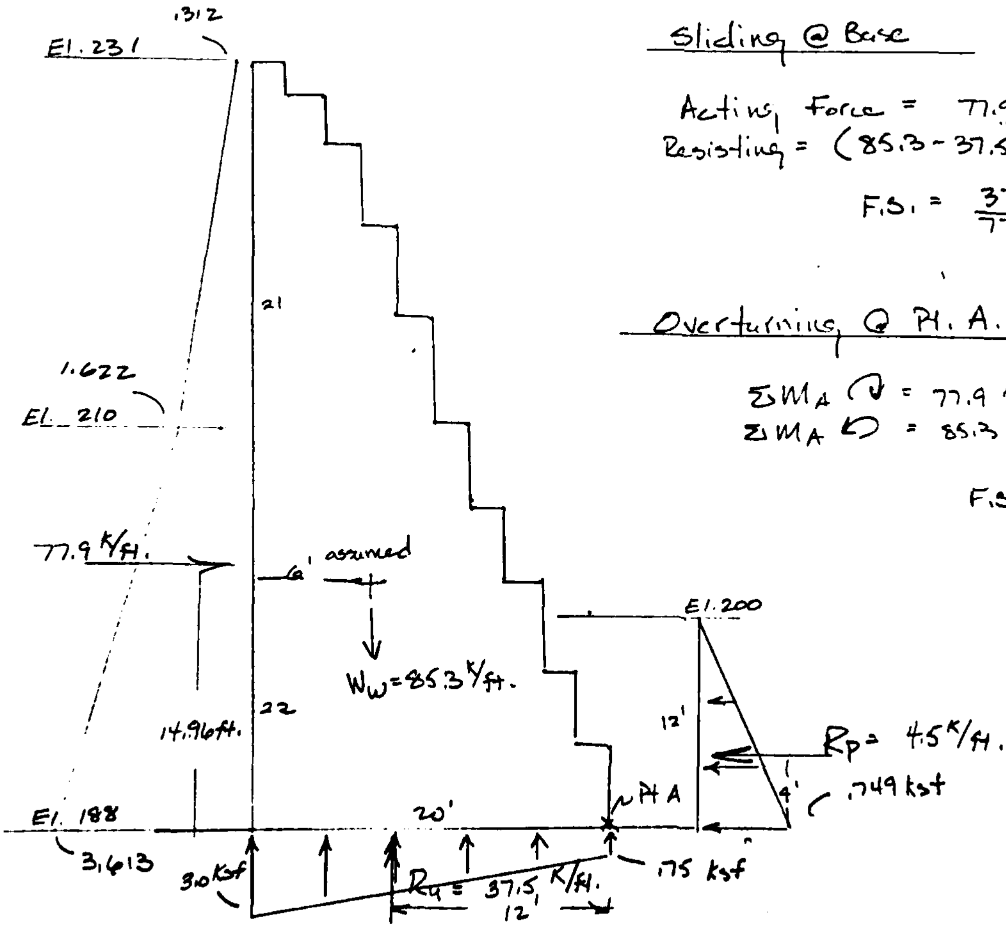
Hydrostatic Pressure (Downstream)

El. 188 = $12 \times .0624 = 0.749$ ksf

Uplift Force

$H_2 = 48 \times .0624 = 3.0$ ksf

$H_1 = 12 \times .0624 = .75$ ksf.



Sliding @ Base

Acting Force = 77.9 k/ft.

Resisting = $(85.3 - 37.5)\,.65 + 4.5 = 35.6$ k/ft.

$$F.S. = \frac{37.6}{77.9} = 0.46$$

Overturning @ Pt. A.

$\Sigma M_A \circlearrowright = 77.9 \times 14.96 + 37.5 \times 12 = 1615$ 'k/'

$\Sigma M_A \circlearrowleft = 85.3 \times 14 + 4.5 \times 4 = 1212$ 'k/'

$$F.S. = \frac{1212}{1615} = 0.75$$

PAD
7.8.79
TED
7/16/78

Analysis No. 4 - Stability of Lake Roland Dam
Seismic Conditions.

SHEET NO. G-9 OF ___

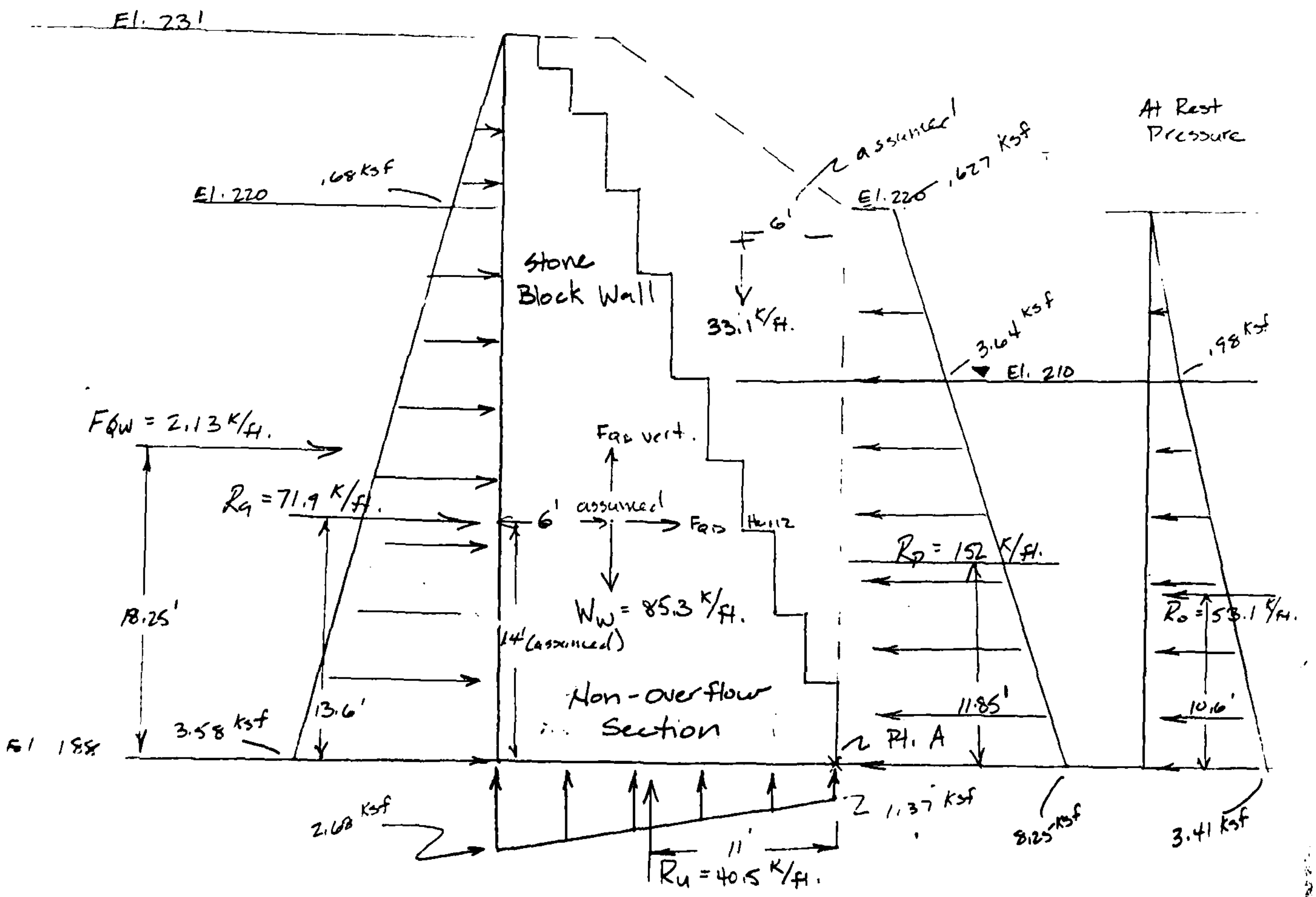
Conditions.

1. Reservoir level @ top of dam
2. Complete soil cover downstream of stone Block wall
3. Seismic Zone 1 horizontal and vertical loading - .025 g.

Assumptions. - See page G-1

Forces on Dam.

Non-seismic forces obtained from analysis No. 2.



PAD
7-8-79
TED
7/16/78

Analysis No. 4 - Stability of Lake Roland Dam.
Seismic Conditions.

SHEET NO G-10CP

Seismic Forces.

Stone Block Wall.

F_{QD} (horiz) = F_{QD} (vert) = $0.025 g \frac{85.3}{32.2} = 2.1$ K/ft

acts at centroid.

Active Force.

F_{QW} (wave) = $\frac{5}{9} \gamma H^2 a/g$.

from: Morris and Wiggert "Applied Hydraulics in Engineering" - pg. 237.

$H = 43$ ft.

use weighted γ to account for sediment.

$\bar{\gamma} = \frac{11}{43} \times 62.4 + \frac{32}{43}(1.45) 62.4 = 83.3$ #/ft³ $= .083$ k/ft³.

$F_{QW} = \frac{5}{9} \times .083$ k/ft³ $\times 43^2 \times .025 g/g = 2.13$ K/ft.

acts @ $\frac{4H}{3\pi}$ from base $= 4/3 \times \frac{43}{\pi} = 18.25$ ft.

Soil Backfill

W = 34 K/ft − .025(34) = 33.1 K/ft.

Passive Force.

Reduce R_p by .025 g/g

$R_p = 152$ K/ft $- .025(152) = 148.2$ K/ft

At Rest Force

Reduce R_o

$R_o = 53.1 - .025(53.1) = 51.8$ K/ft.

Sliding.

Acting Force = 71.9 + 2.13 + 2.1 = 76.1

passive Resisting Force = (85.3 − 40.5 − 2.1 + 33.1) .65 + 148.2 = 197.5

at rest Resisting Force = (85.3 − 40.5 − 2.1 + 33.1) .65 + 51.8 = 101.1

F.S. passive = 2.60 F.S. at rest = 1.33

Overturning.

ΣM_a ↻ = 71.9 × 13.6 + 2.13 × 18.25 + 2.1 × 14 + 40.5 × 11 + 2.1 × 14 = 1521

passive ΣM_a ↺ = (85.3 × 14) + 148.2 × 11.85 + 33.1 × 6 = 3149

at rest ΣM_a ↺ = (85.3 × 14) + 51.8 × 10.6 + 33.1 × 6 = 1942

F.S. passive = $\frac{3149}{1521} = 2.07$ F.S. at rest = $\frac{1942}{1521} = 1.28$

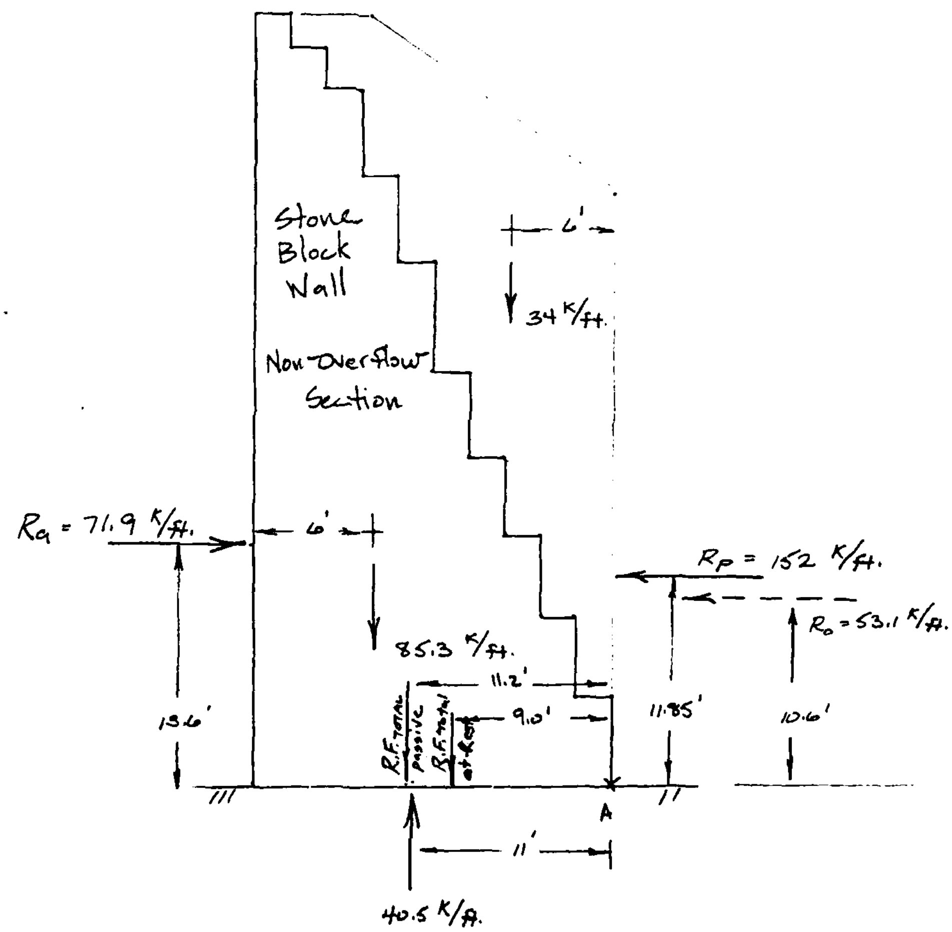
PAD
7/31/79
TED
8/1/79

PROJECT NO

Analysis No. 5 - Location of Resultant Through Dam
Non - Seismic Condition

SHEET NO G-11 OF

Forces Obtained from Analysis # 2



ΣM_a - Passive Condition (+)

ΣM_a = 71.9 × 13.6 + 40.5 × 11 − 85.3 × 14 − 152 × 11.85 − 34 × 6 = −1776

$R.F._{TOTAL}$ = 71.9 + 40.5 − 85.3 − 152 − 34 = −158.9

$\bar{x} = \frac{-1776}{-158.9}$ = 11.2 ft. from pt. A. (middle third)

ΣM_a - At Rest Condition (+)

ΣM_a = 71.9 × 13.6 + 40.5 × 11 − 85.3 × 14 − 53.1 × 10.6 − 34 × 6 = −538

$R.F._{TOTAL}$ = 71.9 + 40.5 − 85.3 − 53.1 − 34 = −60

$\bar{x} = \frac{-538}{-60}$ = 9.0 ft. from pt. A (middle third)